CANADA – NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

Nova Scotia

HYDROTECHNICAL STUDY OF THE TRURO AND AREA FLOODPLAIN

SUMMARY REPORT

MARCH 1988

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

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OF THE
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SUMMARY REPORT

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HYDROTECHNICAL STUDY OF THE TRURO AND AREA FLOODPLAIN

SUMMARY REPORT

1.0 SUMMARY

1.1 Introduction

This report briefly summarizes a technical report prepared for the purpose of delineating flood zones in the Truro area.

1.1.1 The Flood Damage Reduction Program

A floodplain is the flat or gently sloping land adjacent to a river which, during high streamflow conditions caused by excessive rainfall or snowmelt or a combination of the two, becomes inundated by water. Tides and ice also contribute to flooding in many areas of Atlantic Canada. Many communities in Canada developed along rivers and the sea for a number of very good reasons. The nation's waterways were not only the major transportation arteries for the early settlers but, in many instances, served as a source of livelihood. As communities grew, so did occupation of the floodplain to the point where substantial damages, human suffering, and loss of life occurred during major floods.

As flood damages escalated so did pressure to construct flood control works. Significant investments were made in structural works during the 1950's and 1960's. While structural works have been successful in many communities, this approach is not always wise. Structural works are costly to construct and maintain and can only be built to provide a finite level of protection. Because structural works are usually perceived to provide a higher, if not an absolute, level of protection than they are designed for, they usually attract more development on floodplain lands. In some situations, because structures cannot provide absolute protection against all floods, they can be an open invitation to a major catastrophe.

During the mid-1970's, the Federal Government developed the National Flood Damage Reduction Program. The basic objective of the program is to reduce flood damages. Flood Risk Mapping and Studies Agreements were signed between the Province and the Federal Government in 1978 to provide for the identification and delineation of areas prone to flooding in Nova Scotia and to identify and assess remedial works which would reduce flood damages, respectively. These agreements were subsequently extended in 1985 to June 22, 1989.

The Flood Risk Mapping Agreement is the cornerstone of the Flood Damage Reduction Program. Under this Agreement, flood risk areas along a number of watercourses in Nova Scotia are being identified. This information is made available to the general public, to federal, provincial, and municipal government agencies, and to as many potential investors as possible. The flood risk map identifies two zones, the floodway and the floodway fringe. In Nova Scotia, the floodway is defined as that area flooded on an average of once in twenty years or where there is a 5% chance of flooding in any given year. The flood risk area, comprising the floodway and the floodway fringe, includes all that area flooded on an average of once in one hundred years, or where there is a 1% chance of flooding in any given year. Once flood risk maps have been prepared for an area, the federal and provincial governments, through a formal designation process, agree not to engage in or fund developments in the floodway. They will only participate in funding developments in the floodway fringe provided they are adequately floodproofed. Undertakings for agricultural and recreational purposes are excluded from this policy. Furthermore, once an area has been designated, the senior levels of government will withhold disaster assistance payments to developments constructed after designation has been effected.

The Studies Agreement provides for the evaluation of remedial measures to minimize flood damage potential in areas which have a significant flooding problem. Under this agreement, all feasible structural and non-structural measures must be evaluated in terms of effectiveness, cost, benefit, and environmental impact. Structural works include, but are not limited to, flood control dams, dykes, channel improvements and diversions. Non-structural measures include, but are not limited to, zoning, flood warning, emergency measures, land acquisition and watershed management.

This Truro area hydrotechnical study was carried out under the auspices of the Flood Risk Mapping Agreement.

1.1.2 Study Area Description

The area of primary concern in this study is the floodplain lands adjacent to the Salmon River in the vicinity of Truro (upstream to just past Valley Station) and the North River upstream to just above the Community of North River. The limits of the study area are shown on Figure 1.1.

The study area encompasses the Town of Truro, the Village of Bible Hill, and several smaller communities within Colchester County, which collectively have a population of approximately 18,000 people. However, only a relatively small fraction of the total population live within the flood risk areas. Approximately 500-600 buildings are located in the flood risk area and of these, about 300 are residential structures. Therefore, it is estimated that between 1000-2000 residents would be directly affected by the occurrence of a severe flood. There is a substantial land area which, as yet, has not been developed and which also has a high flood potential.

1.1.3 Historical Flooding

The review of historical flooding has confirmed a long history of flood damages in the Truro area. Flooding was found to occur most frequently at 11 flood prone damage centres, which are identified on Figure 1.1. Development in flood prone areas has led to an increased potential for future flood losses.

A number of man-made factors influence both the extent and duration of flood damages which occur during flood events. These factors include dykes, fill and related development, floodplain occupation, bridge construction and gravel removal operations. Any other additional uncontrolled uses of the floodplain in the future would also likely have additional adverse effects in regard to flooding and flood losses.

In addition to man-made factors, a number of natural flood factors have a significant influence on flooding in the Truro area. It was found that heavy rainfall was considered to be the main factor causing serious flooding (about 90% of the cases) while ice jams and snowmelt are mentioned a significant number of times (40 and 45% respectively). The historical records also refer to tides as a factor in flooding for about 15-20% of the recorded events, although no doubt they were a factor in many more. Other natural factors such as scour and deposition of sand, gravel and silt in the lower reaches of the Salmon and North Rivers also contribute to flood conditions, especially those caused by ice jams.

Most of the historical damage centres remain susceptible to future flood damages. However, the magnitude of potential flood damage in the 1980's would be greater than historical flood damages, due simply to the increased value and amount of development that would be affected by flooding. For example, the extensive development of Robie Street in the years since Highway 102 was built will inevitably result in increased flood damage. Uncontrolled additional development could also add significantly to potential flood damages in the future.

1.1.4 Purpose and Scope of the Report

The continued development of flood prone lands has led to a corresponding demand for an increased level of awareness and protection from future flood damages. The purpose of this hydrotechnical study was to provide the basic technical information describing the flood risk in the Truro area. This includes the determination of the elevation of flood waters at various locations and the frequency of occurrence of such floods. This investigation provides the necessary technical information for development of maps to be used for designation of flood risk areas. Such maps can then be used to guide future development patterns and to reduce the potential for increased future flood losses.

The investigations included the following tasks:

1. review of available background data related to historical flood conditions
2. implementation of a field survey program to calibrate and verify the various hydraulic and hydrologic models used
3. preparation of base maps for study purposes and for display of flood risk information
4. assessment of the climatologic, hydrologic, tidal and hydraulic characteristics of the flood prone channels and floodplains in the Truro area
5. determination of water levels resulting from the 1:20 year and 1:100 year flooding conditions and the plotting of flood lines representing these elevations on topographic mapping
6. identification of possible structural and non-structural measures to reduce flood damages for consideration in any future detailed investigation.

1.2 Hydrotechnical Investigations

1.2.1 Background Data

A number of earlier investigations related to the study area were obtained and reviewed for information concerning local flood characteristics. The findings of these investigations tend to confirm the long-term flooding problems which have occurred in the Truro area, as well as confirming the natural and man-made factors affecting flooding as previously identified.

Streamflow data were obtained from Environment Canada at hydrometric stations located within and near the study area. Information on tidal characteristics was obtained from various sources including Fisheries and Oceans Canada and the Nova Scotia Department of Agriculture and Marketing. Information on precipitation was obtained from Environment Canada. This information was used in the determination of design flood conditions for the Truro area.

The evaluation of existing information led to the implementation of a field program to collect additional hydrotechnical data for the study. The need to use complex hydrologic and hydraulic computer modelling techniques was also identified as a result of this assessment.

1.2.2 Field Program

The accurate determination of flooding characteristics required implementation of a field survey program. The following physical information was collected during the field program:

1. width, depth, shape and elevation of the rivers at a number of locations throughout the study area, including the adjacent floodplain

2. a description of the riverbed and adjacent floodplain flow paths in relation to surface roughness, vegetation and shape
3. measurements and physical characteristics of hydraulic structures, bridges, culverts, aboiteaux, and dykes.

The field surveys also included the installation of hydrometric gauging stations in order to obtain water level measurements during peak runoff conditions. Data were collected during the spring of 1980 and 1982. Tidal characteristics were also measured in the field during these events. This provided information for calibrating and verifying the computer models used for calculating flood profiles.

Cross-sections were surveyed along the rivers at intervals of about 1000 metres during the summer of 1982. These cross-sections, together with data describing the various hydraulic structures form the basis for the construction of the hydraulic model used.

A number of significant factors affecting flood conditions were also identified as a result of the field surveys. It was confirmed that the channel bottom in the estuary is subject to wide fluctuations in elevation, location and shape due to sedimentation and erosion. This action results in variations in channel bottom elevations by as much as 2 metres at some locations. It was also confirmed that the dyke elevation controls flood levels at various locations in the study area.

1.2.3 Hydrology

The Salmon and North Rivers, with headwaters in Colchester and Pictou Counties, drain an area of about 726 km^2 into Cobequid Bay. The amount of land inundated by a particular flood depends on the magnitude and volume of the flood event and on the prevailing tidal characteristics in the study area. The impact of channel and floodplain storage and the effects of the dyke system are also significant factors which impact the routing of flood hydrographs in the study area. Ice jamming can also lead to significant localized flooding throughout the study area.

The basic objective of the hydrological analysis was to provide estimates of the 1:20 and 1:100 year design flood hydrographs at specified locations on the Salmon and North Rivers. The design flows were then used for input to the hydraulic model for the determination of water surface profiles.

Separate analytical approaches were used to define the hydrologic regime of the main rivers and of the tributaries. The overall approach follows regional flood frequency procedures previously developed for watersheds in the Province. The design flows on the Salmon and North Rivers were computed by application of regional peak flow prediction equations. A design storm approach using rainfall runoff modelling was adopted for the various tributary streams in the study area. In summary, flow estimates were made for the following locations:

1. by prediction equation technique at -
    - i) Salmon River at Valley Station
    - ii) North River at Stewart Bridge

2. by rainfall runoff modelling method at -
    - i) Mouth of Lepper Brook
    - ii) Mouth of Farnham Brook
    - iii) Mouth of McCurdy Brook
    - iv) Mouth of McClures Brook
    - v) Mouth of Higgin's Brook
    - vi) Mouth of Soley Brook
    - vii) Mouth of Christie Brook

3. by prorating methods for local inflows not covered by 1. and 2. above.

Since tidal effects and the influence of large channel and floodplain storage areas along the study reach have a significant effect on flood levels, it was also necessary to determine appropriate hydrograph volumes and shapes associated with the peak flow estimates.

It was found that the Salmon and North River generally peak at the same time, although brooks within the study area normally peak 4-9 hours earlier than the main rivers. For the purposes of this investigation, the start of the hydrograph rise from the baseflow for the two rivers and tributaries was assumed to be coincident. Sensitivity analyses were also carried out which further confirmed the acceptability of the design flow estimates and their relative timing. The rainfall-runoff model was calibrated using the available data for the tributaries. The calibration and verification runs supported the use of this technique for estimation of design flows for the various tributary brooks.

1.2.4 Tides

The magnitude of tides in the Bay of Fundy is among the highest in the world. Unfortunately, insufficient data were available to directly describe the tidal regime of the study area, and as a consequence, it was necessary to assemble and interpret data from a variety of sources. The tidal gauge at Saint John, New Brunswick has provided the only continuous long-term record of tides in the Bay of Fundy and it was, therefore, utilized as the basis for tidal prediction in the study area. The transfer of tidal characteristics took into account each of the following factors:

1. astronomical factors
2. tidal propagation in the Bay of Fundy
3. atmospheric disturbances
4. coastal submergence along the Bay of Fundy

5. effect of freshwater flows at the downstream boundary of the study area.

The magnitude of design tides (1:20 and 1:100 year recurrence interval design conditions) was determined at the downstream boundary of the study area using the following approach:

1. the assigned frequencies of the astronomic component of the tide at Saint John were determined from existing information
2. a regression relationship predicting tides at Truro as a function of those at Saint John was utilized to transfer the astronomical tide at Saint John to the site of the Old Board Landing Bridge in Truro, based upon coincident records obtained at the two sites in 1971 and 1972
3. a statistical analysis of the effects of meteorological disturbances on the tidal regime at the site of the Old Board Landing Bridge was carried out
4. a joint probability analysis of astronomical and meteorological tides was carried out resulting in estimates of the 1:20 and 1:100 year recurrence interval tide peaks at the site of the Old Board Landing Bridge
5. the design tides at the site of the Old Board Landing Bridge were adjusted further for the effects of coastal submergence
6. the magnitudes of the design tides at the site of the Old Board Landing Bridge were adjusted to the downstream boundary of the study area at McNutt Brook
7. design tidal hydrographs were then determined for the McNutt Brook location by utilizing the classical sinsuoidal tidal variations as recorded at the Burntcoat Head secondary tidal port.

The resulting 1:20 and 1:100 year peak design tide levels at McNutt Brook were determined to be 10.1 m and 10.4 m respectively.

1.2.5 Hydraulics

The main purpose of the project was to calculate the 1:20 and 1:100 year flood profiles along the study area. The computation of water surface profiles and flood hydrograph routing was undertaken by using a computer program referred to as "1-D" developed by Environment Canada, based on methods provided by the Massachusetts Institute of Technology. This model was selected and utilized in order to account for the following hydraulic factors:

1. time varying tidal characteristics at the downstream boundary (Salmon River at McNutt Brook) of the study area
2. streamflow hydrographs used as input at the upstream boundary (Salmon River at Clifford Brook, North River above Stewart Bridge), together with lateral inflows along the study reach. Tributary inflows for Farnham, McCurdy, Christie, Lepper, McClures, Soley and Higgins Brooks were represented by point lateral inflows, with other local runoff represented as uniformly distributed lateral inflow
3. dykes, storage areas and flood channels were all represented by the hydrodynamic model. This included flow through aboiteaux and overtopping of dykes.
4. the relative influence of channel and floodplain conveyance areas
5. other hydraulic factors including structures, head losses due to channel, floodplain conditions, and stream bed conditions.

To ensure reliable simulation results, the hydraulic model was calibrated by adjusting model coefficients and comparing simulated and observed flood profiles along the study reach. The results of the model calibration and subsequent model verification testing indicated that the hydrodynamic simulation model produced acceptable simulation results. The model was then used to simulate different scenarios required for estimating flood levels associated with design events and for undertaking a sensitivity analysis of the effects of variations in hydraulic factors on flood profiles along the study reach.

The sensitivity analysis assessed the relative importance of variations in several key input factors affecting simulation of peak levels. It was found that variations in flood discharge had the greatest effect on simulated profiles, especially for the reaches above the head of tide. The effect of discharge variations tended to reduce gradually as the flood wave proceeded downstream. It was found that tidal effects on peak flood levels were greatest in the downstream reaches, extending up to the vicinity of the C.N.R. bridge on the Salmon River and the Highway 104 bridge on the North River. The effects of flood channel conveyance and storage capabilities were also found to be significant during high flood tide periods. These factors contribute to the attenuation of flood peaks by creating temporary storage of part of the flood volume. A relatively smaller impact on flood levels was found when varying other model parameters.

It was found that the conveyance area required by the 1:20 year flood is adequate to convey the bulk of flood waters for most flood events. Therefore, it is recommended that the 1:20 year flood profile calculated by the 1-D model should be used to determine the areal extent of the floodway within the flood risk area. The 1:100 year flood profile should then be used to determine the extent of the flood fringe area, taking into account the results of the sensitivity testing. By a comparison of simulated and historical flood levels, it was concluded to be appropriate to make an overall adjustment of +0.5 m to the 1:100 year flood profile in order to allow for the effect of numerous factors (e.g. ice jams, etc.) on flood levels. A summary of 1:20 and 1:100 year flood elevations at selected locations is shown in Table 1.1. Also shown is a summary of historical observations taken over the past 30 years. As can be seen, historical flood levels generally fall between estimates of the 1:20 year and 1:100 year flood levels as would be expected.

TABLE 1.1

Summary of Flood Elevations

Location	1:20 Yr. Flood Elevation (m)	1:100 Yr. Flood Elevation (m)	Historical Flood Elevation (m)
Salmon River			
1) At outlet	10.10	10.90	-
2) At Board Landing Br.	10.12	10.84	10.1
3) At confluence with North R.	10.11	10.80	-
4) At Park St. Bridge	10.38	11.26	11.0
5) At CNR Bridge	13.30	14.27	16.0
6) At Murray Siding	23.00	24.01	-
7) At Valley	39.23	40.21	-
North River			
8) Onslow Road Bridge at Marshall's Corner	9.99	10.61	10.8
9) At TransCanada Hwy.	11.89	12.70	-
10) At Stewart Bridge	27.34	28.30	-
11) At Confluence with West North R.	32.19	32.97	-
McClures Brook			
12) D/S (1) of Hwy 102	9.29	10.58	10.0
13) U/S (2) of Hwy 102	9.30	10.52	10.1
14) Upstream of D.A.R.	11.45	12.07	-
Floodplain S. of Salmon River			
15) U/S of Hwy 102 Viaduct	9.36	10.59	10.1
16) South Flood Channel at Park St.	9.28	10.57	-
17) South of North R. Confluence	9.28	10.56	-
Floodplain N. of Salmon River			
18) N. (3) of McCurdy Bk. Aboiteau	9.45	10.43	-
19) N. side of Farnham Bk. Aboiteau	10.20	10.94	-
20) Between Farnham Bk. & Park St.	10.24	10.99	-
21) Between Park St. and CNR	10.38	11.15	10.9
22) McCurdy Brook north of CNR	9.46	10.21	-

Abbreviations

(1) D/S - Downstream
(2) U/S - Upstream
(3) N - North

The 1:20 and 1:100 year flood prone areas were then depicted on the new topographic mapping.

1.3 Possible Remedial Measures

Detailed evaluation of alternative remedial measures for alleviating flooding problems was beyond the scope of the present investigations. However, it has been possible to identify a number of alternative remedial measures for future consideration.

In general, remedial measures can be divided into two main categories as follows:

1. structural measures, which directly affect the flood characteristics, and
2. non-structural measures, which are intended to modify the loss burden either by reducing the potential for continued development in the flood prone lands, or by providing some form of economic relief from flood losses.

Structural Alternatives

A number of potential structural alternatives for area-wide protection and local improvement were identified as summarized on Figure 1.2.

The following summarizes the list of structural alternatives which should be given consideration in any future remedial measures study:

1. local improvements - this should include an investigation of local measures which could be undertaken at each of the flood damage centres. The measures to be assessed include the following:

    i) assessment of the effectiveness of berms in the Cottage/Lepper Brook and in the Riverside/Avon Street areas
    ii) economic assessment and social impact resulting from the relocation of houses in the Montgomery Drive area with a view to undertaking a similar program at other damage centres in the future
    iii) the feasibility of floodproofing existing structures along Robie and Juniper Streets
    iv) the feasibility of local floodproofing in the Upper Onslow, Marshall's Corner and Farnham Road areas.

2. area-wide improvements

   The following area-wide improvements could be investigated in more detail in the future:
    i) raising existing dykes
    ii) construction of new dyke systems
    iii) improvements to existing flood protection and drainage system (straightening of the river, improved drainage from Truro Dykeland Park, provision of flow channel to the McClures Brook aboiteau; and modifications to the berm in the Cottage/Lepper Brook area to create dykes capable of providing protection against both ice jam and open water flooding
3. provision of ice control structures to control movement of ice into the estuary and at other ice jam locations upstream
4. provision of flood control reservoirs in headwater areas
5. construction of a causeway tidal dam.

At the present time it is not considered likely that the economics of the last three structural measures would be proven feasible, due to high capital costs. (This does not imply that any of the other works would be economically feasible.)

Non-structural Alternatives

Since the Truro area is an actively developing area, floodplain regulations should be implemented to guide future development and thus reduce the potential for continued increases in flood damages.

In this case, a two-zone floodway flood-fringe concept is envisaged where zoning regulations would prohibit development in the high risk floodway areas. The floodway would be defined as the extent of the 1:20 year floodplain.

The floodway fringe area would be defined as the extent of the 1:100 year floodplain as outlined on the Flood Risk Maps to be prepared. Additional development would be permitted in the floodway fringe areas, depending on the degree of hazard and the implementation of floodproofing measures to protect these developments. Regulatory agencies should consider adding a freeboard allowance of 0.5 m, however, to account for a multiplicity of factors including basement flooding, ice/debris damage, storm water/sewage backup, etc., etc.

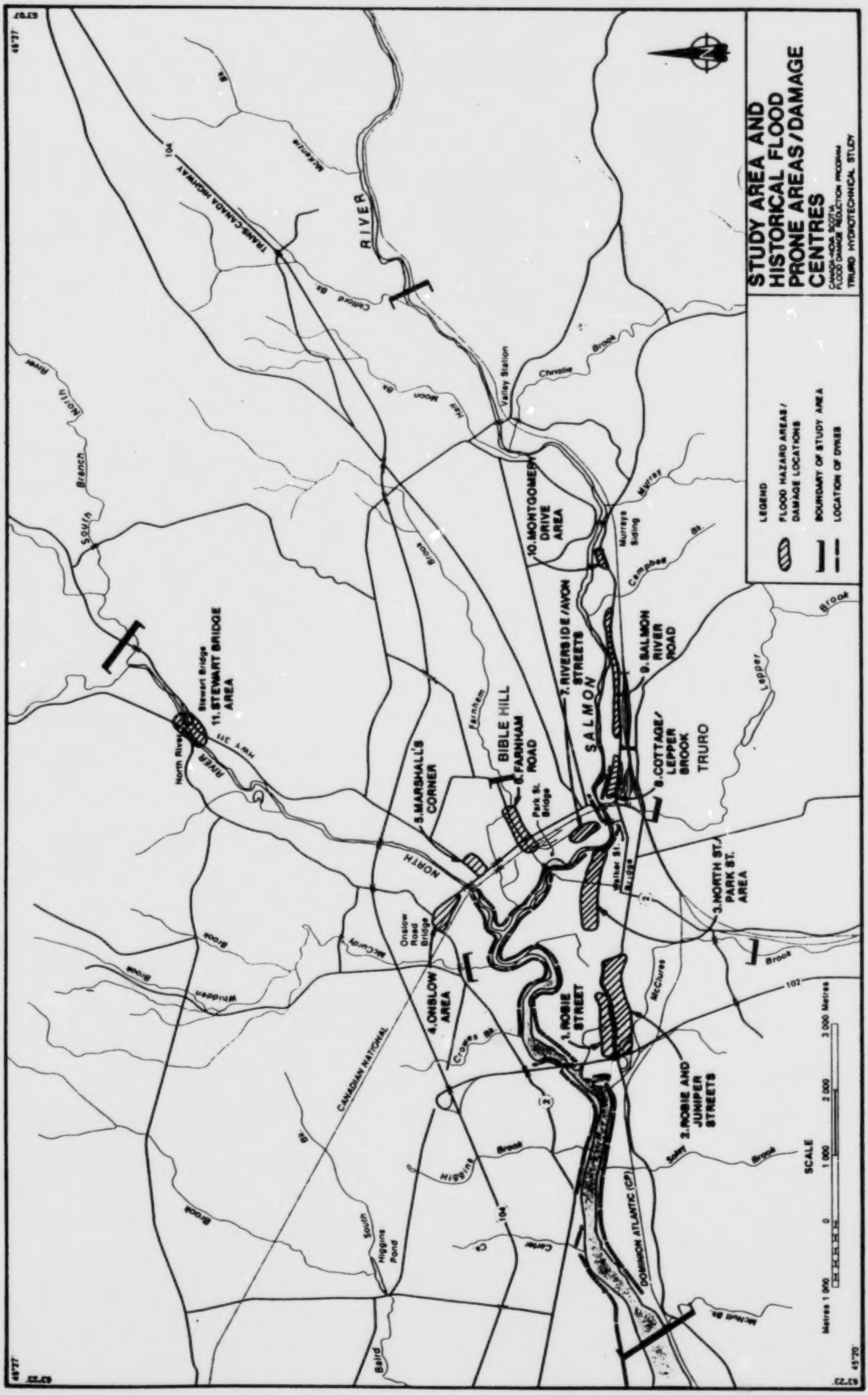
STUDY AREA AND HISTORICAL FLOOD PRONE AREAS/DAMAGE CENTRES
CANADA-NOVA SCOTIA FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY
LEGEND
FLOOD HAZARD AREAS / DAMAGE LOCATIONS
BOUNDARY OF STUDY AREA
LOCATION OF DYKES
SCALE
Metres 1 000 0 1 000 2 000 3 000 Metres
1. ROBIE STREET
2. ROBIE AND JUNIPER STREETS
3. NORTH ST. PARK ST. AREA
4. ONSLOW AREA
5. MARSHALL'S CORNER
6. FARNHAM ROAD
7. RIVERSIDE/AVON STREETS
8. COTTAGE/LEPPER BROOK
9. SALMON RIVER ROAD
10. MONTGOMERY DRIVE AREA
11. STEWART BRIDGE AREA
TRURO
BIBLE HILL
SALMON
RIVER
NORTH
RIVER
North River
South Branch
North River
Stewart Bridge
Hwy 311
Park St. Bridge
Walker St. Bridge
Onslow Road Bridge
Farnham
Brook
McCurdy
Whidden Brook
Brook
Cropes Bk.
CANADIAN NATIONAL
DOMINION ATLANTIC (CP)
TRANS-CANADA HIGHWAY 104
104
102
2
Higgins Brook
South Higgins Pond
Baird
Carter
Brook
Sobey Brook
McClures
Brook
Lepper Brook
Campbell Bk.
Murrays Siding
Murray
Christie Brook
Valley Station
Hay Moon Bk.
Clifford Bk.
McKenzie
Bk.
McNutt Bk.
45°27'
45°20'
63°22'
63°07'

FIGURE 1.1

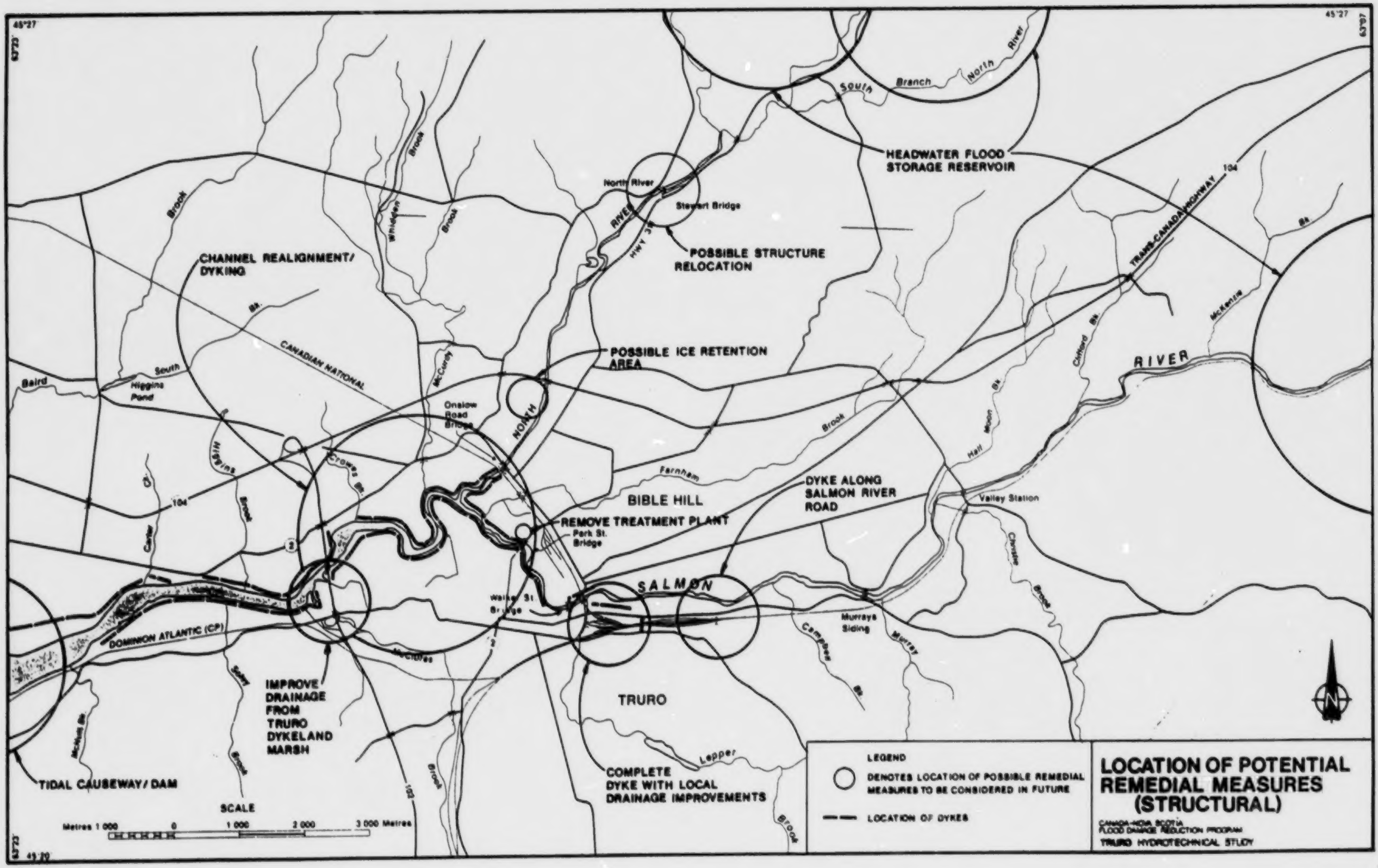
45°27
63°22
45°27
63°07
HEADWATER FLOOD STORAGE RESERVOIR
South Branch North River
North River
Stewart Bridge
RIVER
Hwy 3
POSSIBLE STRUCTURE RELOCATION
CHANNEL REALIGNMENT/ DYKING
Brook
Whidden Brook
Brook
Bk.
CANADIAN NATIONAL
South
Baird
Higgins Pond
McCurdy
Onslow Road Bridge
NORTH
POSSIBLE ICE RETENTION AREA
TRANS-CANADA HIGHWAY
104
Bk.
McKenzie
Clifford Bk.
RIVER
Hall Moon Bk.
Brook
Farnham
BIBLE HILL
DYKE ALONG SALMON RIVER ROAD
Valley Station
REMOVE TREATMENT PLANT
Park St. Bridge
Crowes Bk.
Higgins Brook
104
2
Carter Cr.
Walker St. Bridge
SALMON
Christie Brook
Murrays Siding
Murray
Campbes Bk.
DOMINION ATLANTIC (CP)
McIntires
IMPROVE DRAINAGE FROM TRURO DYKELAND MARSH
TRURO
Salley Brook
McNutt Bk.
102
Brook
COMPLETE DYKE WITH LOCAL DRAINAGE IMPROVEMENTS
Lepper Brook
TIDAL CAUSEWAY/ DAM
SCALE
Metres 1 000 0 1 000 2 000 3 000 Metres
63°22
45°20
N
LEGEND
DENOTES LOCATION OF POSSIBLE REMEDIAL MEASURES TO BE CONSIDERED IN FUTURE
LOCATION OF DYKES
LOCATION OF POTENTIAL REMEDIAL MEASURES (STRUCTURAL)
CANADA-NOVA SCOTIA FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 1.2

CANADA - NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

Nova Scotia

HYDROTECHNICAL STUDY OF THE TRURO AND AREA FLOODPLAIN

MARCH 1988

HYDROTECHNICAL STUDY OF THE TRURO AND AREA FLOODPLAIN

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HYDROTECHNICAL STUDY OF THE TRURO AND AREA FLOODPLAIN

1.0 SUMMARY

1.1 Introduction

This report briefly summarizes a technical report prepared for the purpose of delineating flood zones in the Truro area.

1.1.1 The Flood Damage Reduction Program

A floodplain is the flat or gently sloping land adjacent to a river which, during high streamflow conditions caused by excessive rainfall or snowmelt or a combination of the two, becomes inundated by water. Tides and ice also contribute to flooding in many areas of Atlantic Canada. Many communities in Canada developed along rivers and the sea for a number of very good reasons. The nation's waterways were not only the major transportation arteries for the early settlers but, in many instances, served as a source of livelihood. As communities grew, so did occupation of the floodplain to the point where substantial damages, human suffering, and loss of life occurred during major floods.

As flood damages escalated so did pressure to construct flood control works. Significant investments were made in structural works during the 1950's and 1960's. While structural works have been successful in many communities, this approach is not always wise. Structural works are costly to construct and maintain and can only be built to provide a finite level of protection. Because structural works are usually perceived to provide a higher, if not an absolute, level of protection than they are designed for, they usually attract more development on floodplain lands. In some situations, because structures cannot provide absolute protection against all floods, they can be an open invitation to a major catastrophe.

During the mid-1970's, the Federal Government developed the National Flood Damage Reduction Program. The basic objective of the program is to reduce flood damages. Flood Risk Mapping and Studies Agreements were signed between the Province and the Federal Government in 1978 to provide for the identification and delineation of areas prone to flooding in Nova Scotia and to identify and assess remedial works which would reduce flood damages, respectively. These agreements were subsequently extended in 1985 to June 22, 1989.

The Flood Risk Mapping Agreement is the cornerstone of the Flood Damage Reduction Program. Under this Agreement, flood risk areas along a number of watercourses in Nova Scotia are being identified. This information is made available to the general public, to federal, provincial, and municipal government agencies, and to as many potential investors as possible. The flood risk map identifies two zones, the floodway and the floodway fringe. In Nova Scotia, the floodway is defined as that area flooded on an average of once in twenty years or where there is a 5% chance of flooding in any given year. The flood risk area, comprising the floodway and the floodway fringe, includes all that area flooded on an average of once in one hundred years, or where there is a 1% chance of flooding in any given year. Once flood risk maps have been prepared for an area, the federal and provincial governments, through a formal designation process, agree not to engage in or fund developments in the floodway. They will only participate in funding developments in the floodway fringe provided they are adequately floodproofed. Undertakings for agricultural and recreational purposes are excluded from this policy. Furthermore, once an area has been designated, the senior levels of government will withhold disaster assistance payments to developments constructed after designation has been effected.

The Studies Agreement provides for the evaluation of remedial measures to minimize flood damage potential in areas which have a significant flooding problem. Under this agreement, all feasible structural and non-structural measures must be evaluated in terms of effectiveness, cost, benefit, and environmental impact. Structural works include, but are not limited to, flood control dams, dykes, channel improvements and diversions. Non-structural measures include, but are not limited to, zoning, flood warning, emergency measures, land acquisition and watershed management.

This Truro area hydrotechnical study was carried out under the auspices of the Flood Risk Mapping Agreement.

1.1.2 Study Area Description

The area of primary concern in this study is the floodplain lands adjacent to the Salmon River in the vicinity of Truro (upstream to just past Valley Station) and the North River upstream to just above the Community of North River. The limits of the study area are shown on Figure 1.1.

The study area encompasses the Town of Truro, the Village of Bible Hill, and several smaller communities within Colchester County, which collectively have a population of approximately 18,000 people. However, only a relatively small fraction of the total population live within the flood risk areas. Approximately 500-600 buildings are located in the flood risk area and of these, about 300 are residential structures. Therefore, it is estimated that between 1000-2000 residents would be directly affected by the occurrence of a severe flood. There is a substantial land area which, as yet, has not been developed and which also has a high flood potential.

1.1.3 Historical Flooding

The review of historical flooding has confirmed a long history of flood damages in the Truro area. Flooding was found to occur most frequently at 11 flood prone damage centres, which are identified on Figure 1.1. Development in flood prone areas has led to an increased potential for future flood losses.

A number of man-made factors influence both the extent and duration of flood damages which occur during flood events. These factors include dykes, fill and related development, floodplain occupation, bridge construction and gravel removal operations. Any other additional uncontrolled uses of the floodplain in the future would also likely have additional adverse effects in regard to flooding and flood losses.

In addition to man-made factors, a number of natural flood factors have a significant influence on flooding in the Truro area. It was found that heavy rainfall was considered to be the main factor causing serious flooding (about 90% of the cases) while ice jams and snowmelt are mentioned a significant number of times (40 and 45% respectively). The historical records also refer to tides as a factor in flooding for about 15-20% of the recorded events, although no doubt they were a factor in many more. Other natural factors such as scour and deposition of sand, gravel and silt in the lower reaches of the Salmon and North Rivers also contribute to flood conditions, especially those caused by ice jams.

Most of the historical damage centres remain susceptible to future flood damages. However, the magnitude of potential flood damage in the 1980's would be greater than historical flood damages, due simply to the increased value and amount of development that would be affected by flooding. For example, the extensive development of Robie Street in the years since Highway 102 was built will inevitably result in increased flood damage. Uncontrolled additional development could also add significantly to potential flood damages in the future.

1.1.4 Purpose and Scope of the Report

The continued development of flood prone lands has led to a corresponding demand for an increased level of awareness and protection from future flood damages. The purpose of this hydrotechnical study was to provide the basic technical information describing the flood risk in the Truro area. This includes the determination of the elevation of flood waters at various locations and the frequency of occurrence of such floods. This investigation provides the necessary technical information for development of maps to be used for designation of flood risk areas. Such maps can then be used to guide future development patterns and to reduce the potential for increased future flood losses.

The investigations included the following tasks:

1. review of available background data related to historical flood conditions
2. implementation of a field survey program to calibrate and verify the various hydraulic and hydrologic models used
3. preparation of base maps for study purposes and for display of flood risk information
4. assessment of the climatologic, hydrologic, tidal and hydraulic characteristics of the flood prone channels and floodplains in the Truro area
5. determination of water levels resulting from the 1:20 year and 1:100 year flooding conditions and the plotting of flood lines representing these elevations on topographic mapping
6. identification of possible structural and non-structural measures to reduce flood damages for consideration in any future detailed investigation.

1.2 Hydrotechnical Investigations

1.2.1 Background Data

A number of earlier investigations related to the study area were obtained and reviewed for information concerning local flood characteristics. The findings of these investigations tend to confirm the long-term flooding problems which have occurred in the Truro area, as well as confirming the natural and man-made factors affecting flooding as previously identified.

Streamflow data were obtained from Environment Canada at hydrometric stations located within and near the study area. Information on tidal characteristics was obtained from various sources including Fisheries and Oceans Canada and the Nova Scotia Department of Agriculture and Marketing. Information on precipitation was obtained from Environment Canada. This information was used in the determination of design flood conditions for the Truro area.

The evaluation of existing information led to the implementation of a field program to collect additional hydrotechnical data for the study. The need to use complex hydrologic and hydraulic computer modelling techniques was also identified as a result of this assessment.

1.2.2 Field Program

The accurate determination of flooding characteristics required implementation of a field survey program. The following physical information was collected during the field program:

1. width, depth, shape and elevation of the rivers at a number of locations throughout the study area, including the adjacent floodplain

2. a description of the riverbed and adjacent floodplain flow paths in relation to surface roughness, vegetation and shape
3. measurements and physical characteristics of hydraulic structures, bridges, culverts, aboiteaux, and dykes.

The field surveys also included the installation of hydrometric gauging stations in order to obtain water level measurements during peak runoff conditions. Data were collected during the spring of 1980 and 1982. Tidal characteristics were also measured in the field during these events. This provided information for calibrating and verifying the computer models used for calculating flood profiles.

Cross-sections were surveyed along the rivers at intervals of about 1000 metres during the summer of 1982. These cross-sections, together with data describing the various hydraulic structures form the basis for the construction of the hydraulic model used.

A number of significant factors affecting flood conditions were also identified as a result of the field surveys. It was confirmed that the channel bottom in the estuary is subject to wide fluctuations in elevation, location and shape due to sedimentation and erosion. This action results in variations in channel bottom elevations by as much as 2 metres at some locations. It was also confirmed that the dyke elevation controls flood levels at various locations in the study area.

1.2.3 Hydrology

The Salmon and North Rivers, with headwaters in Colchester and Pictou Counties, drain an area of about 726 km^2 into Cobequid Bay. The amount of land inundated by a particular flood depends on the magnitude and volume of the flood event and on the prevailing tidal characteristics in the study area. The impact of channel and floodplain storage and the effects of the dyke system are also significant factors which impact the routing of flood hydrographs in the study area. Ice jamming can also lead to significant localized flooding throughout the study area.

The basic objective of the hydrological analysis was to provide estimates of the 1:20 and 1:100 year design flood hydrographs at specified locations on the Salmon and North Rivers. The design flows were then used for input to the hydraulic model for the determination of water surface profiles.

Separate analytical approaches were used to define the hydrologic regime of the main rivers and of the tributaries. The overall approach follows regional flood frequency procedures previously developed for watersheds in the Province. The design flows on the Salmon and North Rivers were computed by application of regional peak flow prediction equations. A design storm approach using rainfall runoff modelling was adopted for the various tributary streams in the study area. In summary, flow estimates were made for the following locations:

1. by prediction equation technique at -
    i) Salmon River at Valley Station
    ii) North River at Stewart Bridge

2. by rainfall runoff modelling method at -
    i) Mouth of Lepper Brook
    ii) Mouth of Farnham Brook
    iii) Mouth of McCurdy Brook
    iv) Mouth of McClures Brook
    v) Mouth of Higgin's Brook
    vi) Mouth of Soley Brook
    vii) Mouth of Christie Brook

3. by prorating methods for local inflows not covered by 1. and 2. above.

Since tidal effects and the influence of large channel and floodplain storage areas along the study reach have a significant effect on flood levels, it was also necessary to determine appropriate hydrograph volumes and shapes associated with the peak flow estimates.

It was found that the Salmon and North River generally peak at the same time, although brooks within the study area normally peak 4-9 hours earlier than the main rivers. For the purposes of this investigation, the start of the hydrograph rise from the baseflow for the two rivers and tributaries was assumed to be coincident. Sensitivity analyses were also carried out which further confirmed the acceptability of the design flow estimates and their relative timing. The rainfall-runoff model was calibrated using the available data for the tributaries. The calibration and verification runs supported the use of this technique for estimation of design flows for the various tributary brooks.

1.2.4 Tides

The magnitude of tides in the Bay of Fundy is among the highest in the world. Unfortunately, insufficient data were available to directly describe the tidal regime of the study area, and as a consequence, it was necessary to assemble and interpret data from a variety of sources. The tidal gauge at Saint John, New Brunswick has provided the only continuous long-term record of tides in the Bay of Fundy and it was, therefore, utilized as the basis for tidal prediction in the study area. The transfer of tidal characteristics took into account each of the following factors:

1. astronomical factors
2. tidal propagation in the Bay of Fundy
3. atmospheric disturbances
4. coastal submergence along the Bay of Fundy

5. effect of freshwater flows at the downstream boundary of the study area.

The magnitude of design tides (1:20 and 1:100 year recurrence interval design conditions) was determined at the downstream boundary of the study area using the following approach:

1. the assigned frequencies of the astronomic component of the tide at Saint John were determined from existing information
2. a regression relationship predicting tides at Truro as a function of those at Saint John was utilized to transfer the astronomical tide at Saint John to the site of the Old Board Landing Bridge in Truro, based upon coincident records obtained at the two sites in 1971 and 1972
3. a statistical analysis of the effects of meteorological disturbances on the tidal regime at the site of the Old Board Landing Bridge was carried out
4. a joint probability analysis of astronomical and meteorological tides was carried out resulting in estimates of the 1:20 and 1:100 year recurrence interval tide peaks at the site of the Old Board Landing Bridge
5. the design tides at the site of the Old Board Landing Bridge were adjusted further for the effects of coastal submergence
6. the magnitudes of the design tides at the site of the Old Board Landing Bridge were adjusted to the downstream boundary of the study area at McNutt Brook
7. design tidal hydrographs were then determined for the McNutt Brook location by utilizing the classical sinsuoidal tidal variations as recorded at the Burntcoat Head secondary tidal port.

The resulting 1:20 and 1:100 year peak design tide levels at McNutt Brook were determined to be 10.1 m and 10.4 m respectively.

1.2.5 Hydraulics

The main purpose of the project was to calculate the 1:20 and 1:100 year flood profiles along the study area. The computation of water surface profiles and flood hydrograph routing was undertaken by using a computer program referred to as "1-D" developed by Environment Canada, based on methods provided by the Massachusetts Institute of Technology. This model was selected and utilized in order to account for the following hydraulic factors:

1. time varying tidal characteristics at the downstream boundary (Salmon River at McNutt Brook) of the study area
2. streamflow hydrographs used as input at the upstream boundary (Salmon River at Clifford Brook, North River above Stewart Bridge), together with lateral inflows along the study reach. Tributary inflows for Farnham, McCurdy, Christie, Lepper, McClures, Soley and Higgins Brooks were represented by point lateral inflows, with other local runoff represented as uniformly distributed lateral inflow
3. dykes, storage areas and flood channels were all represented by the hydrodynamic model. This included flow through aboiteaux and overtopping of dykes.
4. the relative influence of channel and floodplain conveyance areas
5. other hydraulic factors including structures, head losses due to channel, floodplain conditions, and stream bed conditions.

To ensure reliable simulation results, the hydraulic model was calibrated by adjusting model coefficients and comparing simulated and observed flood profiles along the study reach. The results of the model calibration and subsequent model verification testing indicated that the hydrodynamic simulation model produced acceptable simulation results. The model was then used to simulate different scenarios required for estimating flood levels associated with design events and for undertaking a sensitivity analysis of the effects of variations in hydraulic factors on flood profiles along the study reach.

The sensitivity analysis assessed the relative importance of variations in several key input factors affecting simulation of peak levels. It was found that variations in flood discharge had the greatest effect on simulated profiles, especially for the reaches above the head of tide. The effect of discharge variations tended to reduce gradually as the flood wave proceeded downstream. It was found that tidal effects on peak flood levels were greatest in the downstream reaches, extending up to the vicinity of the C.N.R. bridge on the Salmon River and the Highway 104 bridge on the North River. The effects of flood channel conveyance and storage capabilities were also found to be significant during high flood tide periods. These factors contribute to the attenuation of flood peaks by creating temporary storage of part of the flood volume. A relatively smaller impact on flood levels was found when varying other model parameters.

It was found that the conveyance area required by the 1:20 year flood is adequate to convey the bulk of flood waters for most flood events. Therefore, it is recommended that the 1:20 year flood profile calculated by the 1-D model should be used to determine the areal extent of the floodway within the flood risk area. The 1:100 year flood profile should then be used to determine the extent of the flood fringe area, taking into account the results of the sensitivity testing. By a comparison of simulated and historical flood levels, it was concluded to be appropriate to make an overall adjustment of +0.5 m to the 1:100 year flood profile in order to allow for the effect of numerous factors (e.g. ice jams, etc.) on flood levels. A summary of 1:20 and 1:100 year flood elevations at selected locations is shown in Table 1.1. Also shown is a summary of historical observations taken over the past 30 years. As can be seen, historical flood levels generally fall between estimates of the 1:20 year and 1:100 year flood levels as would be expected.

TABLE 1.1

Summary of Flood Elevations

Location	1:20 Yr. Flood Elevation (m)	1:100 Yr. Flood Elevation (m)	Historical Flood Elevation (m)
Salmon River			
1) At outlet	10.10	10.90	-
2) At Board Landing Br.	10.12	10.84	10.1
3) At confluence with North R.	10.11	10.80	-
4) At Park St. Bridge	10.38	11.26	11.0
5) At CNR Bridge	13.30	14.27	16.0
6) At Murray Siding	23.00	24.01	-
7) At Valley	39.23	40.21	-
North River			
8) Onslow Road Bridge at Marshall's Corner	9.99	10.61	10.8
9) At TransCanada Hwy.	11.89	12.70	-
10) At Stewart Bridge	27.34	28.30	-
11) At Confluence with West North R.	32.19	32.97	-
McClures Brook			
12) D/S (1) of Hwy 102	9.29	10.58	10.0
13) U/S (2) of Hwy 102	9.30	10.52	10.1
14) Upstream of D.A.R.	11.45	12.07	-
Floodplain S. of Salmon River			
15) U/S of Hwy 102 Viaduct	9.36	10.59	10.1
16) South Flood Channel at Park St.	9.28	10.57	-
17) South of North R. Confluence	9.28	10.56	-
Floodplain N. of Salmon River			
18) N. (3) of McCurdy Bk. Aboiteau	9.45	10.43	-
19) N. side of Farnham Bk. Aboiteau	10.20	10.94	-
20) Between Farnham Bk. & Park St.	10.24	10.99	-
21) Between Park St. and CNR	10.38	11.15	10.9
22) McCurdy Brook north of CNR	9.46	10.21	-

Abbreviations

(1) D/S - Downstream
(2) U/S - Upstream
(3) N - North

The 1:20 and 1:100 year flood prone areas were then depicted on the new topographic mapping.

1.3 Possible Remedial Measures

Detailed evaluation of alternative remedial measures for alleviating flooding problems was beyond the scope of the present investigations. However, it has been possible to identify a number of alternative remedial measures for future consideration.

In general, remedial measures can be divided into two main categories as follows:

1. structural measures, which directly affect the flood characteristics, and
2. non-structural measures, which are intended to modify the loss burden either by reducing the potential for continued development in the flood prone lands, or by providing some form of economic relief from flood losses.

Structural Alternatives

A number of potential structural alternatives for area-wide protection and local improvement were identified as summarized on Figure 1.2.

The following summarizes the list of structural alternatives which should be given consideration in any future remedial measures study:

1. local improvements - this should include an investigation of local measures which could be undertaken at each of the flood damage centres. The measures to be assessed include the following:

- i) assessment of the effectiveness of berms in the Cottage/Lepper Brook and in the Riverside/Avon Street areas
- ii) economic assessment and social impact resulting from the relocation of houses in the Montgomery Drive area with a view to undertaking a similar program at other damage centres in the future
- iii) the feasibility of floodproofing existing structures along Robie and Juniper Streets
- iv) the feasibility of local floodproofing in the Upper Onslow, Marshall's Corner and Farnham Road areas.

2. area-wide improvements

   The following area-wide improvements could be investigated in more detail in the future:
   - i) raising existing dykes
   - ii) construction of new dyke systems
   - iii) improvements to existing flood protection and drainage system (straightening of the river, improved drainage from Truro Dykeland Park, provision of flow channel to the McClures Brook aboiteau; and modifications to the berm in the Cottage/Lepper Brook area to create dykes capable of providing protection against both ice jam and open water flooding
3. provision of ice control structures to control movement of ice into the estuary and at other ice jam locations upstream
4. provision of flood control reservoirs in headwater areas
5. construction of a causeway tidal dam.

At the present time it is not considered likely that the economics of the last three structural measures would be proven feasible, due to high capital costs. (This does not imply that any of the other works would be economically feasible.)

Non-structural Alternatives

Since the Truro area is an actively developing area, floodplain regulations should be implemented to guide future development and thus reduce the potential for continued increases in flood damages.

In this case, a two-zone floodway flood-fringe concept is envisaged where zoning regulations would prohibit development in the high risk floodway areas. The floodway would be defined as the extent of the 1:20 year floodplain.

The floodway fringe area would be defined as the extent of the 1:100 year floodplain as outlined on the Flood Risk Maps to be prepared. Additional development would be permitted in the floodway fringe areas, depending on the degree of hazard and the implementation of floodproofing measures to protect these developments. Regulatory agencies should consider adding a freeboard allowance of 0.5 m, however, to account for a multiplicity of factors including basement flooding, ice/debris damage, storm water/sewage backup, etc., etc.

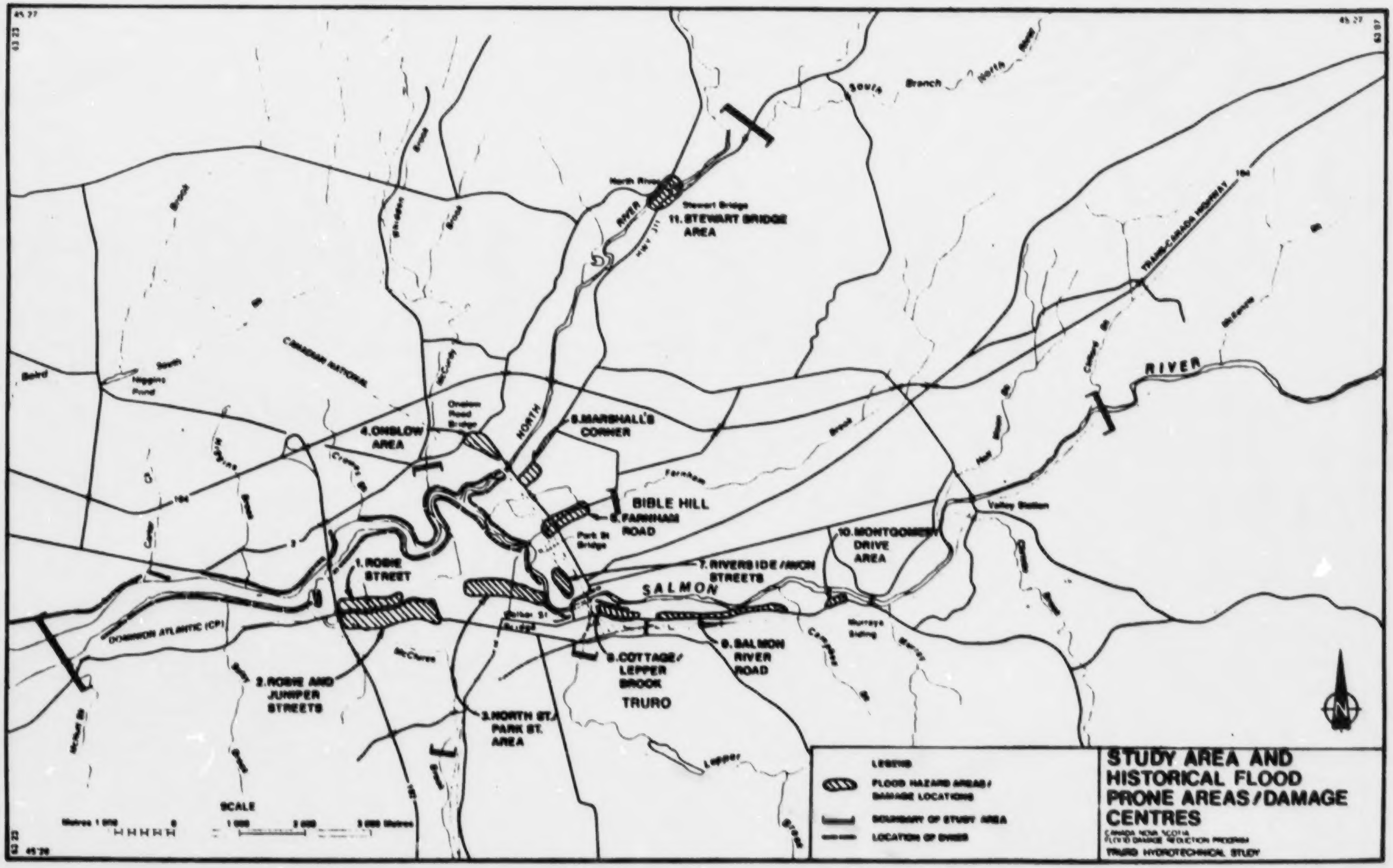

STUDY AREA AND HISTORICAL FLOOD PRONE AREAS/DAMAGE CENTRES
CANADA NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY
LEGEND
FLOOD HAZARD AREAS/DAMAGE LOCATIONS
BOUNDARY OF STUDY AREA
LOCATION OF DIKES
SCALE
Metres 1 000
0
1 000
2 000
3 000 Metres
1.ROBIE STREET
2.ROBIE AND JUNIPER STREETS
3.NORTH ST./PARK ST. AREA
4.ONSLOW AREA
5.COTTAGE/LEPPER BROOK
6.FARNHAM ROAD
7.RIVERSIDE/INCH STREETS
8.SALMON RIVER ROAD
9.MARSHALL'S CORNER
10.MONTGOMERY DRIVE AREA
11.STEWART BRIDGE AREA
BIBLE HILL
TRURO
SALMON
RIVER
NORTH
RIVER
North River
Stewart Bridge
HWY 311
Onslow Road Bridge
Park St Bridge
Valley Station
Murrays Siding
DOMINION ATLANTIC (CP)
CANADIAN NATIONAL
TRANS-CANADA HIGHWAY
104
102
South Branch
North River
Farnham
Higgins Pond
Lepper
Brook
McClures
Baird
45 27
63 23
45 20
63 07

FIGURE 1.1

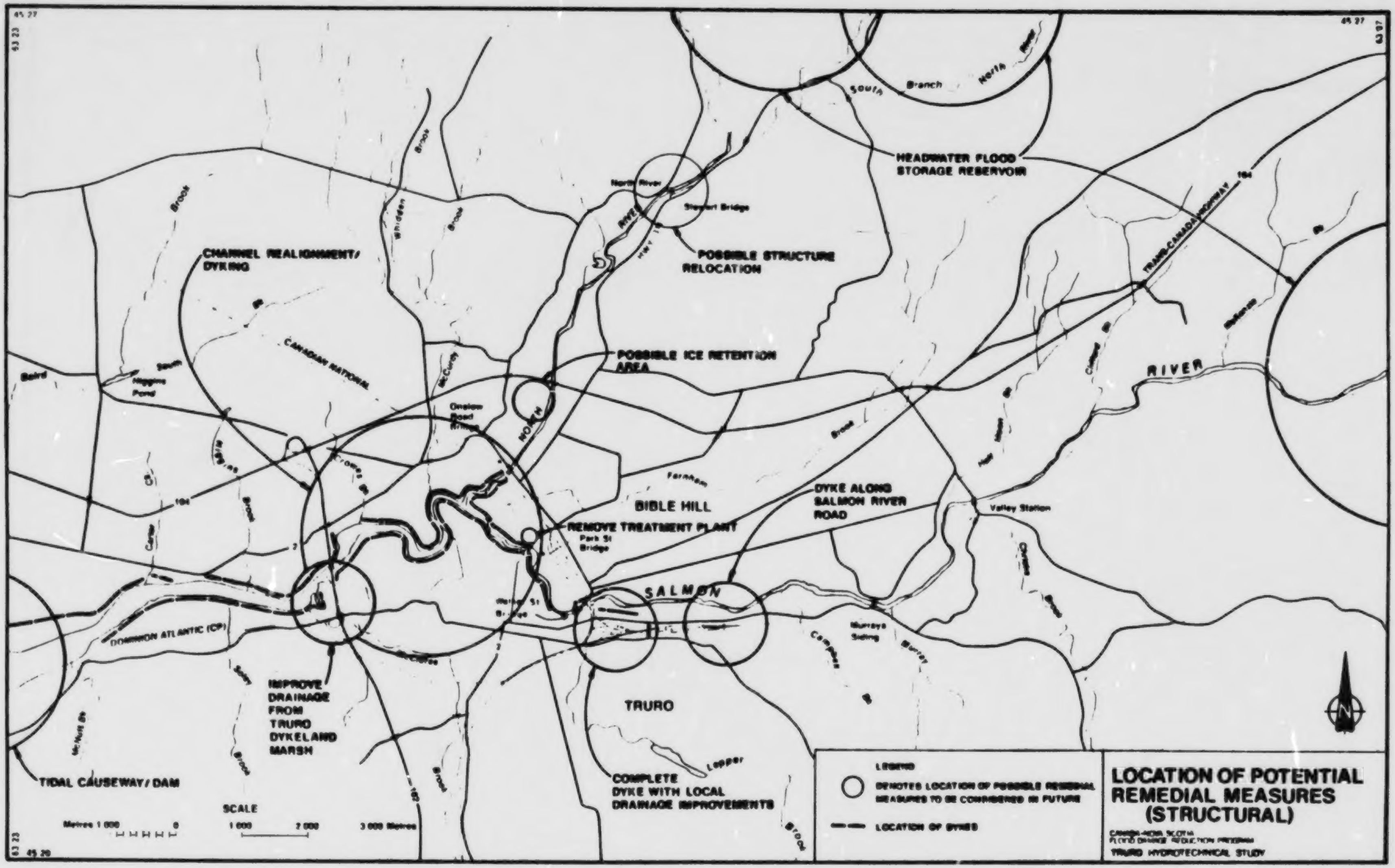

LOCATION OF POTENTIAL
REMEDIAL MEASURES
(STRUCTURAL)
CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY
LEGEND
DENOTES LOCATION OF POSSIBLE REMEDIAL MEASURES TO BE CONSIDERED IN FUTURE
LOCATION OF DYKES
HEADWATER FLOOD STORAGE RESERVOIR
POSSIBLE STRUCTURE RELOCATION
CHANNEL REALIGNMENT/ DYKING
POSSIBLE ICE RETENTION AREA
REMOVE TREATMENT PLANT
DYKE ALONG SALMON RIVER ROAD
IMPROVE DRAINAGE FROM TRURO DYKELAND MARSH
COMPLETE DYKE WITH LOCAL DRAINAGE IMPROVEMENTS
TIDAL CAUSEWAY/ DAM
BIBLE HILL
TRURO
SALMON
RIVER
NORTH
North River
Stewart Bridge
South
Branch
North
Onslow Road Bridge
Park St Bridge
Walker St Bridge
Farnham
Valley Station
Murrays Siding
TRANS-CANADA HIGHWAY
CANADIAN NATIONAL
DOMINION ATLANTIC (CP)
Whidden Brook
McCurdy
Higgins Pond
Baird
Lepper
Brook
SCALE
Metres 1 000 0 1 000 2 000 3 000 Metres
45 27
45 20
63 23
63 07

FIGURE 1.2

2.0 INTRODUCTION

2.1 General

The Town of Truro and Village of Bible Hill are situated at the head of tide on the Salmon River. The general location of the study area is shown on Figure 2.1. The community was originally established at this location because of the utility of the river and estuary for water supply and transportation. The community has grown to be an important service centre to the agricultural industry. Because road and rail transport facilities must pass through or near it, the community also serves major centres within and outside the province.

Farming has been important to the economy of the area for more than two hundred years. The floodplain lands near the river have been developed for agriculture by dyking the tidal marshes. This practice was started by the original Acadian settlers and subsequently continued by English settlers after expulsion of the Acadians around 1785. For as long as people have lived there, the risk of flooding has existed and residents have been more or less aware of the potential for flood losses.

There is increasing pressure to use the marshland for purposes other than farming. For example, transportation corridors such as the Trans Canada Highway (#104) and the Halifax Connector (#102) have been constructed on the marshland. The commercial uses that were subsequently attracted to develop adjacent to these main highways are likewise subjected to a flood potential.

It is of interest to note that the oldest remaining structures near the river are usually found on high ground at elevations above frequent flood waters. Some of the newer structures have been placed on slightly raised ground, while some others have been sited at very low elevations relative to flood levels. As a consequence, numerous developments have suffered flood damages, and governments have been asked to take action to prevent or reduce the potential for increased flood losses.

A number of floods have produced significant damage in recent years. Historical flooding conditions are discussed in more detail in Chapter 3.0. Past floods have caused direct loss in the form of damaged structures and contents, in addition to broad social and commercial disruption due to road closures, and other indirect factors.

2.2 Flood Damage Reduction Program

On June 22, 1978, the Province of Nova Scotia and the Government of Canada entered into a "General Agreement Respecting Flood Damage Reduction" with the objective of reducing flood damage on floodplains along the shores of lakes, rivers and the sea. This agreement recognizes that the potential for flood damage can be reduced by controlling the uses of land prone to flooding. This agreement was subsequently extended on September 22, 1984 to June 22, 1994.

To provide for the identification and delineation of such flood prone areas in Nova Scotia, the "Agreement Respecting Flood Risk Mapping" was also signed on June 22, 1978. Under the terms of this agreement, the flood risk areas are to be mapped at a number of communities and delineated as areas wherein only certain conforming development should take place. This Agreement was also extended on September 22, 1984 and expires on June 22, 1989.

These agreements provide for the establishment of a Steering Committee and a Flood Risk Mapping Technical Committee. The Steering Committee provides overall coordination, direction and supervision of projects under the agreements. The Technical Committee reports to the Steering Committee and provides technical expertise, makes recommendations with respect to specific programs and undertakes to implement approved projects either with government staff or by supervising consultants or contractors.

In order to ensure that the program is carried out to appropriate technical standards which are uniform across the country, technical guidelines have been developed for both the hydrologic/hydraulic and topographic mapping components of the program (Environment Canada, 1976). The Technical Committee has adhered to these technical guidelines as closely as possible.

2.3 Purpose of This Study

The first step towards achieving the overall program objectives is the definition of the nature and extent of the flood hazard. This includes the determination of the elevation of flood waters at various locations and the frequency of occurrence of such floods.

This investigation provides the necessary technical information for development of maps to be used for designation of flood hazard areas. These Flood Risk Maps can then be used to control future development patterns and reduce the potential for increased future flood losses.

2.4 Scope of This Study

Conceptually, the flood risk mapping program consists of four major components: hydrology, hydraulics, topographic mapping and public information. The hydrologic component involves the determination of the response of a watershed to major climatic events such as rainstorms, rapid snowmelt or a combination of both. The output from the hydrologic component, in the form of flood flows for specified probabilities, serves as the major input to the hydraulic component. The hydraulic component defines the response of the river reaches under consideration to the hydrologic input and other relevant factors. The results of the hydraulic studies, in the form of water surface profiles, are applied to detailed topographic maps to delineate the areal extent of flooding. The final component involves the development of maps, brochures, and other interpretative information for the purposes of informing the public, government agencies and private companies of the flood hazard.

This study encompasses the following major steps:

1. preparation of base maps for study purposes, and for display of flood risk information
2. examination of the hydrology and hydraulics of the Salmon and North River watershed, including the Salmon River estuary to determine its behaviour, and in particular, water levels during high and extreme flood events
3. plotting flood water elevations on the topographic base maps so as to show the areas at risk of flooding from the various design storm events.

The hydrologic component is described in Chapter 6.0 and the hydraulic investigation is described in Chapter 8.0 of this report. Subsequent to this investigation, public information maps will be prepared and disseminated.

Heavy rainfall, snowmelt, tides and ice jams all contribute to the flooding problem in the Truro Area. These factors are discussed in more detail in Chapters 3.0 and 4.0 of this report, together with a summary of existing data and previous investigations. A description of tidal characteristics is presented in Chapter 7.0. A complex flooding regime exists in the study area due to the interrelationships among the various factors affecting flood conditions.

This investigation examines these and other factors and their interaction in order to determine the mechanics of flooding, and the elevations and extent of flooding that occurs or will occur on the average of once in twenty years (1:20) and once in one hundred years (1:100) in the study area.

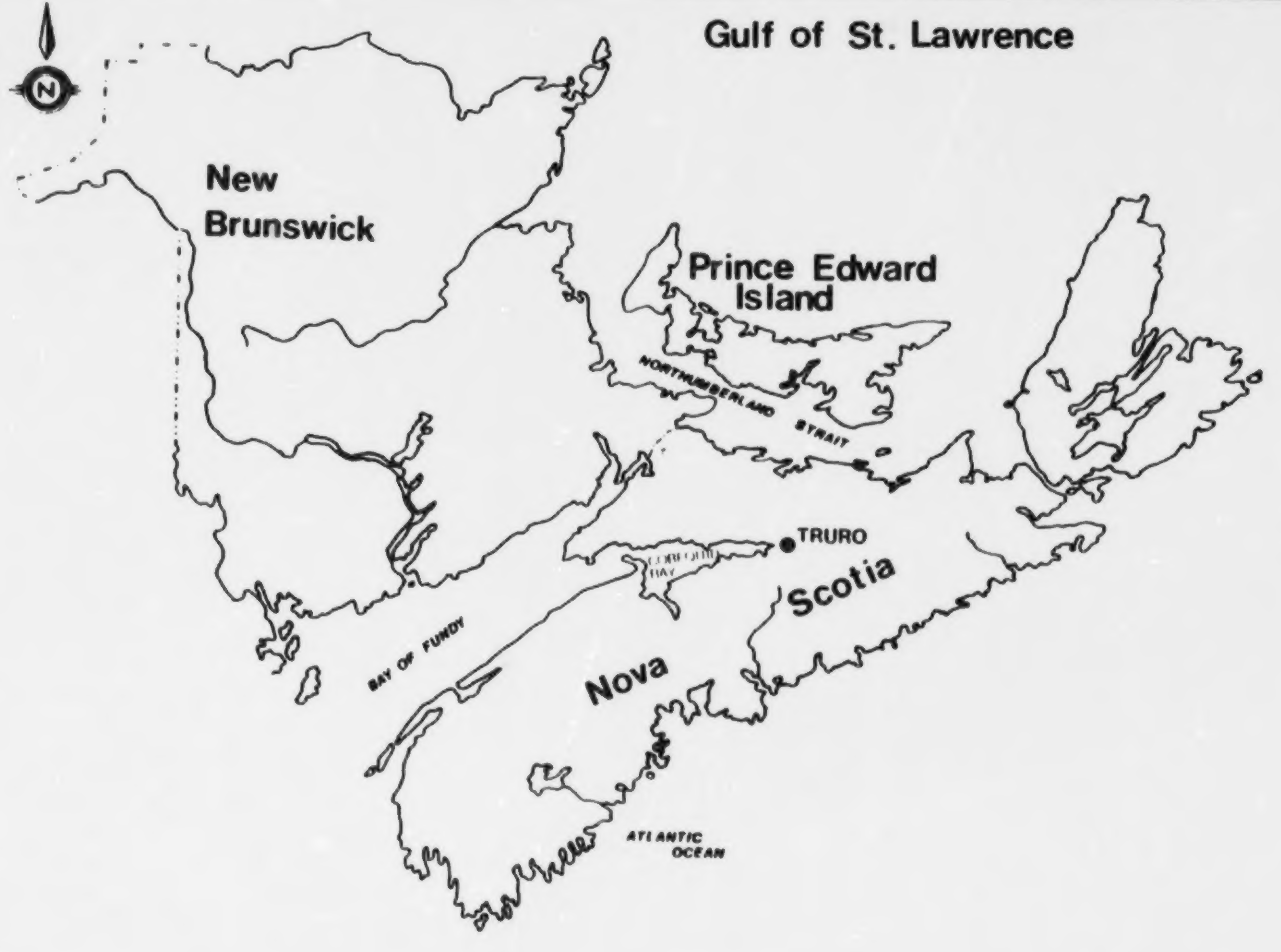
2.5 Study Area Description

The Salmon River has its headwaters in Colchester and Pictou Counties and flows southerly for about 24 km and then westerly for 24 km, entering Cobequid Bay near Truro. The North River is its major tributary, joining the Salmon near Upper Onslow.

The area of primary concern in this study is the land adjacent to the Salmon River beginning in Lower Truro where McNutt Brook discharges and proceeding upstream past Valley Station to where Clifford Brook discharges. Areas along the North River from its confluence with the Salmon River to just beyond its confluence with the West North River and North River are also included. Likewise it includes McClures Brook upstream to McClure's Mills and the lower portions of Farnham and Lepper Brook downstream from Vimy Road and Brunswick Street respectively. The limits of the study area are shown on Figure 2.2. (Street names are shown on Figure 2.3.)

The study area encompasses the Town of Truro, the Village of Bible Hill and several other communities within Colchester County which collectively have a population of approximately 18,000 people. However, only a relatively small fraction of the total population live within flood risk areas. Approximately 500-600 buildings are located in the flood risk area and of these about 300 are residential structures. Therefore, it is estimated that between 1000 to 2000 residents would be directly affected by the occurrence of a severe flood.

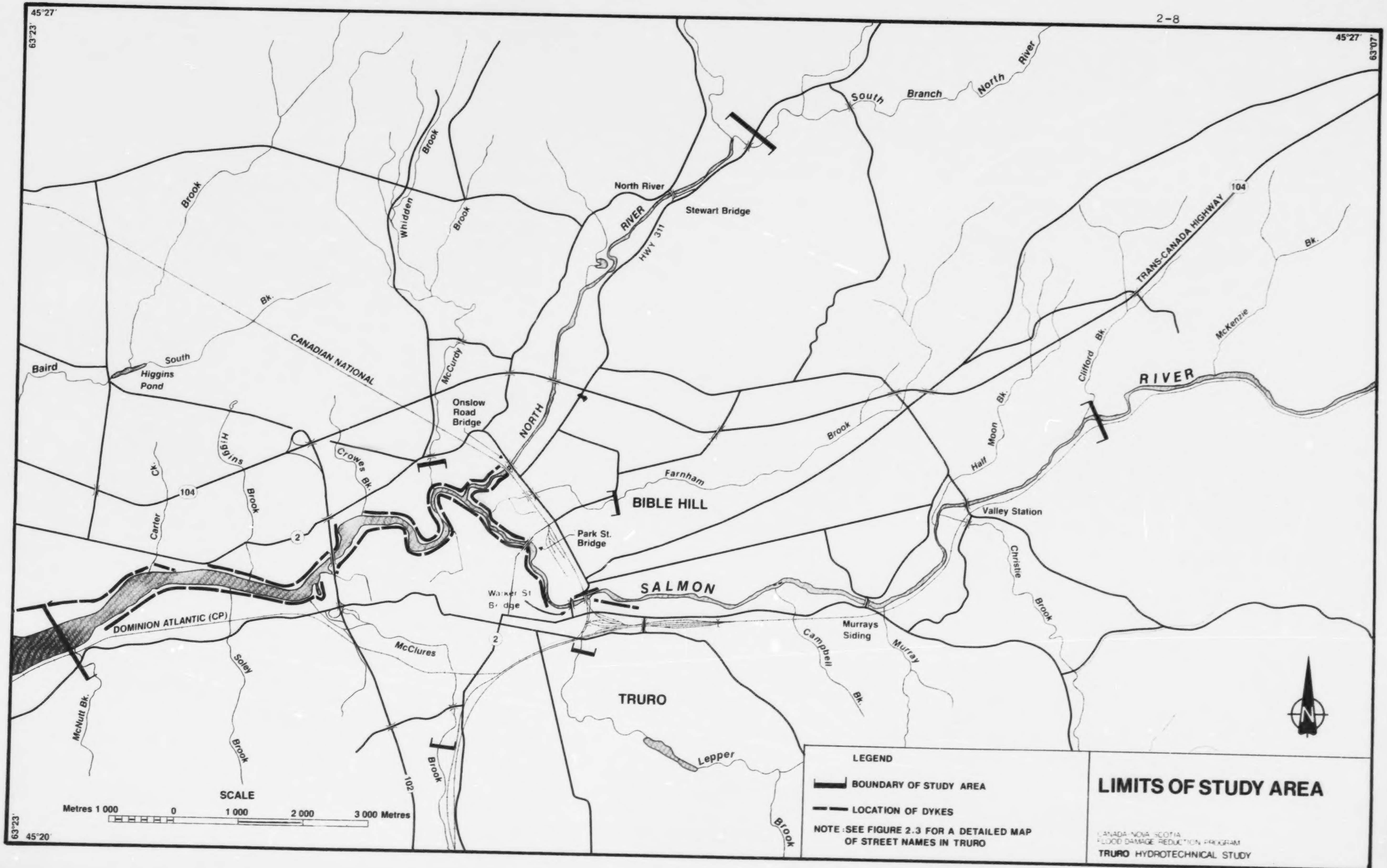
From the lower study limit upstream to the general vicinity of the Walker Street bridge, the tidal effects are significant. The corresponding limit of tidal influence on the North River is near the TransCanada Highway bridge near Upper Onslow. Along these reaches, a system of dykes has been constructed which is primarily intended to confine high tides to the river channel. Beyond these points, dykes and berms have also been constructed to give a measure of protection against flooding resulting from freshwater flooding and ice jam events. However, these dykes and berms provide only limited protection.


Gulf of St. Lawrence
New Brunswick
Prince Edward Island
NORTHUMBERLAND STRAIT
TRURO
COBEQUID BAY
BAY OF FUNDY
Nova Scotia
ATLANTIC OCEAN
N
SCALE
30 0 30 km

GENERAL LOCATION OF STUDY AREA

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 2.1


45°27'
63°23'
45°27'
63°07'
South Branch North River
North River
Stewart Bridge
RIVER
HWY 311
Whidden Brook
McCurdy Brook
Brook
Bk.
CANADIAN NATIONAL
Baird
South
Higgins Pond
Higgins Brook
Carter Ck.
104
2
Crowes Bk.
Onslow Road Bridge
NORTH
Farnham Brook
BIBLE HILL
Park St. Bridge
Walker St Bridge
SALMON
RIVER
TRANS-CANADA HIGHWAY
104
McKenzie
Bk.
Clifford Bk.
Half Moon Bk.
Valley Station
Christie Brook
Murrays Siding
Murray
Campbell Bk.
DOMINION ATLANTIC (CP)
McClures
Soley Brook
McNutt Bk.
Brook
102
Brook
TRURO
Lepper Brook
N
SCALE
Metres 1 000
0
1 000
2 000
3 000 Metres
63°23'
45°20'

LEGEND

- ▬ BOUNDARY OF STUDY AREA
- ━ ━ LOCATION OF DYKES

NOTE : SEE FIGURE 2.3 FOR A DETAILED MAP OF STREET NAMES IN TRURO

LIMITS OF STUDY AREA

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 2.2

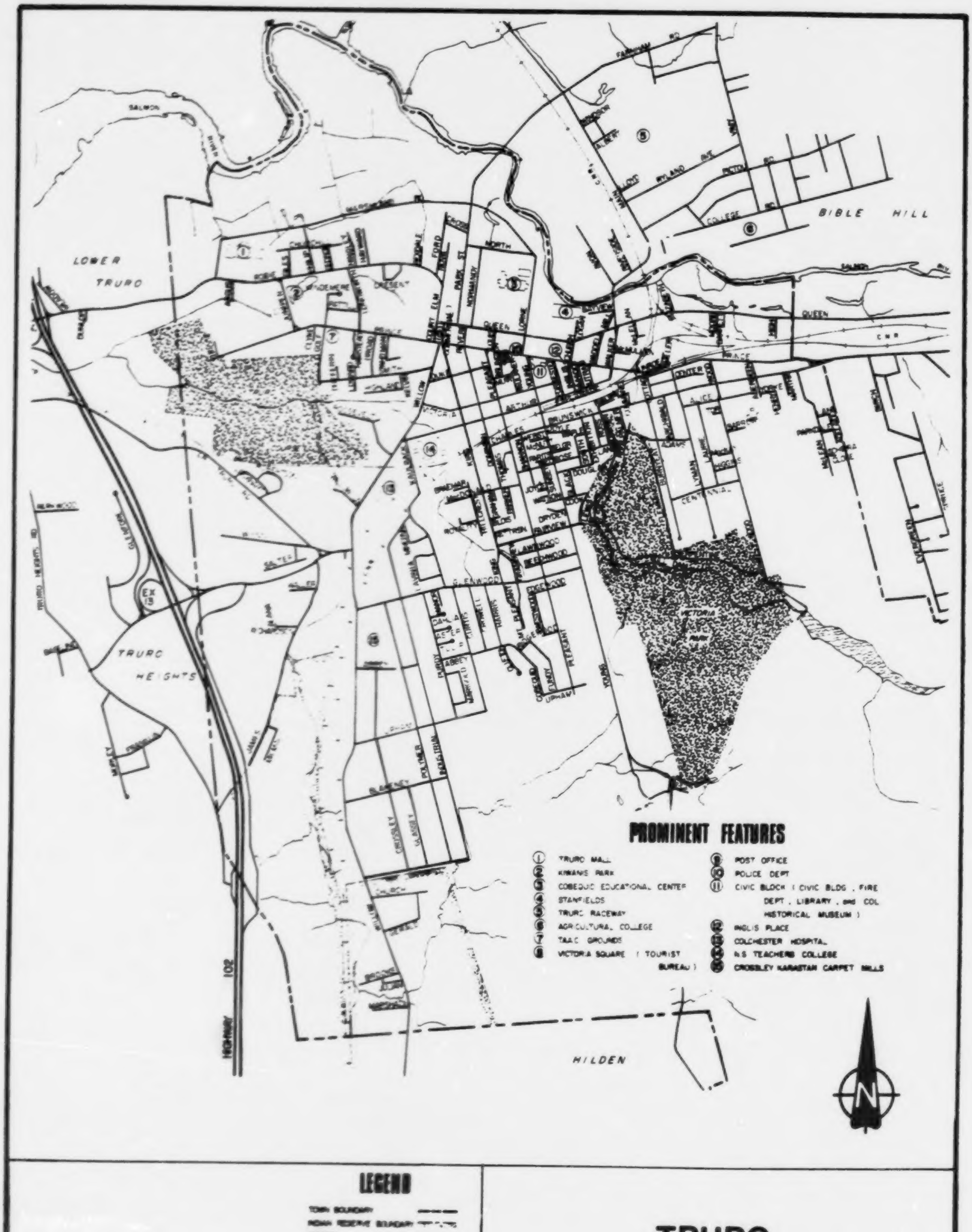

SALMON RIVER
LOWER TRURO
BIBLE HILL
TRURO HEIGHTS
HILDEN
HIGHWAY 102
QUEEN
PRINCE
CNR
VICTORIA PARK
PROMINENT FEATURES
① TRURO MALL
② KIWANIS PARK
③ COBEQUID EDUCATIONAL CENTER
④ STANFIELDS
⑤ TRURO RACEWAY
⑥ AGRICULTURAL COLLEGE
⑦ T.A.A.C. GROUNDS
⑧ VICTORIA SQUARE (TOURIST BUREAU)
⑨ POST OFFICE
⑩ POLICE DEPT
⑪ CIVIC BLOCK (CIVIC BLDG , FIRE DEPT , LIBRARY , and COL HISTORICAL MUSEUM)
⑫ INGLIS PLACE
⑬ COLCHESTER HOSPITAL
⑭ N.S TEACHERS COLLEGE
⑮ CROSSLEY KARASTAN CARPET MILLS
N
LEGEND
TOWN BOUNDARY
INDIAN RESERVE BOUNDARY
STREET
RAILWAY
BROOK
MAJOR RECREATION AREA
RESERVOIR
CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY
TRURO
SCALE
FEET

FIGURE 2.3

3.0 MAGNITUDE AND CAUSES OF FLOODING

3.1 General

It is difficult to reach definitive conclusions about the exact nature of the causative factors involved in past flood events. The existing records describing past events are not clear, although it is apparent that several factors have usually combined to cause significant flood levels. A summary of historical flood events is given in Section 3.2. The usual factors of heavy rainfall and rainfall/snowmelt conditions have been present in most cases. Additional factors in many cases include ice jamming and extreme tides. The natural factors affecting flooding in the study area are discussed in greater detail in Section 3.3.1.

It is also clear that over time, man-made impediments to natural flow patterns have been introduced in the study area. These physical features also contribute to flood damages by interfering with natural flow characteristics and preventing flood flows from discharging to natural storage areas. The man-made factors which contribute to flood losses in the study area are discussed in Section 3.3.2.

An attempt to summarize the main factors and to put the historical and future flood losses in perspective is found in Section 3.4.

3.2 Summary of Historical Flood Events

Newspapers, reports, and documents prepared by various agencies describe a number of flooding events in the Salmon River and North River basins. A list containing information such as the date, location, cause and a brief description of each event is presented in Table 3.1. This information was extracted from "Flooding Events in Nova Scotia, An Historical Perspective", and various other documents which are on file with the Water Planning and Management Branch, Inland Waters/Lands Directorate. In addition to the

TABLE 3.1
Historical Summary of Flooding in Truro

Date	Cause	Description/Location
September 1792	Rain	Interval and marsh between Truro and Onslow was flooded.
October 1869	Saxby Gale, high tides, high winds	Streets in Truro completely flooded. Post Road to Londonderry was impassable.
August 1873	Heavy rain accompanied by strong winds & high tides	The flooding in the Truro area was reported to be worse than the "Saxby Gale" of 1869.
April 1906		Marshes were flooded, damage to dykes.
December 1906	High temperatures, rain, melting snow and ice jams	Salmon River overflowed its banks, surrounding several homes on Water Street. Several homes near Subway had their basements inundated. Flooding also occurred on Ford, Robie and lower Park Street. All the marshes at Lower Truro and in Onslow were submerged.
February 1908	Rain, mild temperatures, ice jams	Flooding occurred in Avon Street area.
March 1920	Wind, rain & high temperatures causing snowmelt	Salmon River flooded the marsh; ice jam on Lepper Brook at Priest Bridge caused flooding of CNR yard, Brunswick, Young & Arthur Streets; Riverside and interval homes were flooded to first floor; 30 families were evacuated from homes on Ford Street; MacMillan Dam at Union was reported to have gone out.
October 1923	Tropical storm, high winds and heavy rain	Stewarts Bridge over North River and Valley Bridge over Salmon River were carried out; Onslow Station near Bible Hill racetrack was under 0.6 m of water; dykes on left bank of Salmon River near Board Landing bridge were broken; road from Board Landing bridge to corner of Lornedale Ave. & Robie Street was flooded; in Bible Hill, the lumber plant owned by D.C. MacKenzie was flooded; Subway flooded to a 0.6 m depth.
October 1927	Heavy rains, high tides	Onslow Road near North River Bridge & portion beyond was covered with water; south bank between Walker Street bridge and railway bridge was flooded.

TABLE 3.1 (cont'd)

Date	Cause	Description/Location
September 1929	Heavy rains	In the North River and Onslow district, marshes and roads were flooded.
April 1931	Rain, high tides	Salmon & North Rivers overflowed inundating the main highway from Bible Hill to Upper Onslow.
October 1933	Extremely high rainfall	Salmon & North Rivers overflowed their banks; road to Lower Truro and road along the flats in Bible Hill were both under water.
November 1934	Heavy rain	Road flooded to a depth of 0.6 m or more in North River area; Salmon and North Rivers were running bank-high.
January 1935	Unusually high temperatures, torrential rains,	Lowlands flooded to a 3.0 m depth; Salmon River overflowed below Walker Street bridge to rear of Flemming Ave.; ice jams on Lepper Brook causing flooding on Cottage Street. Road to Londonderry was impassable.
March 1936	Heavy rains, melting snow, ice jams	Ice jam at Board Landing bridge caused flooding of highway at southern end of bridge; ice jam at railway bridge upstream of Walker Street bridge; highway at Murray Siding bridge washed out.
September 1936	High winds, heavy rains	Low-lying land at Truro inundated as brooks & rivers overflowed their banks; water over road in many places of Lower Truro; highway construction suffered a serious washout at Kemptown.
September 1946	Heavy rainfall for 18 hours or more	Town reservoir was overflowing after a shortage of water; news reported that recent dredging apparently controlled flooding in Salmon River district; marshes in Lower Truro and Onslow district were inundated.
May 1947	Torrential rains, nearly five inches in three days	All low-lying districts around town were flooded; bridge near Bent's Garage in Lower Truro was carried away; above railway bridge, land flooded to Queen Street; subway flooded to a depth of 0.3 m; an aboiteau near Board Landing bridge was washed out.

TABLE 3.1 (cont'd)

Date	Cause	Description/Location
May 1947	Heavy showers	Marshlands and low farmlands were inundated; at Lower Truro, highway bridge was washed out; Subway flooded to a .45 m depth.
February 1948	High temperatures resulting in ice jams	Ice jams on North River; highways from Truro to Tatamagouche & Central North River inundated.
August 1949	Heavy rains, high winds	Marshes were inundated when Salmon and North Rivers overflowed their banks. Several streets in Truro were also inundated due to the overtopping of the storm sewers.
January 1952	Heavy rains, ice jams, high tides	Lowlands were flooded; ice jam at the confluence; Avon Street, Bible Hill & Upper Water Street were flooded; pilings for Stanfield's retaining wall were broken off near elevation of 12 m. Geodetic; 0.6 m of water on Route 2 south of Board Landing Bridge.
February 1953	Heavy rains, high temperatures, snowmelt	Main Street, Riverside Ave. & Avon Street were all flooded; Subway was flooded.
January 1956	High temperatures, heavy rains, ice jams	Board Landing Bridge toward Upper Onslow was inundated; Churchill St. Marsh was covered with water & ice; ice jam east of Dickie Bros. farm in Onslow caused water to back up near Maritime Auto Salvage & Palliser Restaurant; Shipley's Lane, Ford Street, Park St. extension, Dunlop & Robie Streets were flooded; in Onslow district, the Fundy Farms' Dairy store was isolated and school was cancelled.
January 1957	Heavy rain, high tides	North River overflowed its banks in Onslow area inundating fields; Subway flooded to a 0.90 m depth.
March 1957	Heavy rain, mild temperatures, ice jams	North River overflowed its banks in the Onslow area covering highway with water and ice; some minor flooding near Salmon River Bridge (Walker Street).

TABLE 3.1 (cont'd)

Date	Cause	Description/Location
April 1958	Spring thaw with exceptionally high tides	Dykes in Truro area withheld the onslaught of the tides, except for one at Masstown which overflowed.
November 1959	Heavy rainfall	Very little flooding.
April 1962	Heavy rainfall, melting snow	Marsh flooded with approximately .6-.9 m of water in Moose Creek area up to Juniper Street; Salmon River topped dykes downstream from Park St. bridge to confluence; Dykeland Park, Robie St. cemetery and Loughead's Transfer Shed were all inundated; Subway flooded; highway from Bible Hill to Central Onslow flooded to a depth of approx. 0.6 m
August 1963	Heavy rains	Low-lying areas flooded.
April 1964	Heavy rains, high tides	Overbank flooding caused basement flooding in low-lying areas; Subway flooded to a 0.3 m depth.
October 1967	Heavy rains	Rivers high; no mention of flooding; rainfall record to date was broken at Bible Hill with 67.6 mm in a 24-hr. period.
February 1971	Heavy rains, high temperatures, ice jams	Ice jam at C.N.R. bridge on North River; aboiteau on McCurdy Brook forced water and ice flows onto lowest Onslow-North River tract; extensive area of Truro flooded.
August 1971	Record rainfall, Hurricane Beth (11.67" at Halifax Int'l Airport	Salmon River overflowed flooding marshes; Park St. bridge to CNR level crossing was covered with water; several homes in Ford St. area had flooded basements; Subway was flooded.
February 1972	Heavy rains, ice jams	Golf Street area flooded; Park St. bridge & Upper Onslow flooded; Montgomery Drive flood resulted because of an ice jam occurring upstream; Subway was flooded.

TABLE 3.1 (cont'd)

Date	Cause	Description/Location
March 1972	Heavy rain, ice jams	Montgomery Drive, Park Street and Marshall's Corner were all flooded; Lepper Brook overflowed banks when an ice jam occurred at CNR bridge behind Halifax Wholesalers Ltd., Subway & Truro Raceway were flooded.
February 1973	Mild temperature, ice jams	Montgomery Drive flooded; roads washed out in Marshall's Corner, Park St. & Upper Onslow areas.
February 1974	Heavy rain, ice jams	North & Salmon Rivers broke up and ice jams caused overbank flooding; highways in Truro, North River, Onslow & Salmon River were flooded; marshes and numerous basements were inundated; water on lawn of Cobequid Educational Centre.
March 1975	Heavy rain, ice jams	Significant flooding in Christie Brook watershed; ice jam below railroad bridge near Riversdale Lumber yard; on the upstream side of Murray Bridge major slumping occurred on north bank; main channel blocked by ice jams so water flowed outside of channel; ice jams upstream of cheese factory; Montgomery Dr. was flooded; over-the-road flooding occurred between the community of Salmon River & Truro; ice jam behind N.S. Agriculture College; ice jam on Lepper Brook upstream of confluence; highways flooded in Lower & Upper Onslow & North River Road; McClures Brook overflowed its banks; Subway was flooded.
December 1975	Rain, melting snow	Flooding of lower Robie Street business area occurred.
January 1978	Heavy rain	Robie Street business district inundated, some places with as much as 0.9 m of water; schools closed; many roads and streets were inundated and damaged; many residents of low areas bordering Salmon River and in Bible Hill and Lower Truro were forced to leave their homes; hundreds of basements were reported flooded.

TABLE 3.1 (cont'd)

Date	Cause	Description/Location
January 1978	Heavy rain, high temperatures, melting snow	Roads closed; Robie Street business area flooded; homes flooded in Bible Hill.
January 1978	High winds, heavy rain and melting snow	Several streets in and around Truro were inundated. These included Golf St. and the parking lot at the Country Club as well as a portion of the Robie Street business district. Some homes in the Bible Hill and Marshall's Corner areas reported flooded basements.
January 1979	Rain, mild temperatures, ice jams	Ice jammed behind Stanfield's around 4:30 p.m. January 2; Subway inundated for short period; ice cakes deposited on Holy Well Park; Park St. inundated with water and ice north of bridge; marsh body between Salmon & North Rivers inundated with .3-.6 m of water on morning of January 3.
January 1979	Rain, mild temperatures, high tides & ice jams	Ice jammed near the Farmers' Cheese plant at Salmon River; January 29, jam broke on its own; Park St. & Subway inundated; marsh body between Salmon & North River inundated.
February 1979	Rain, mild temperatures, ice jams & high tides	McClures Brook high, flooding most of parking lot of Fundy Trail Mall and Princeport Interchange for short time; water entered B & M Service to a depth of .15-.30 m; water covered portions of golf course for a short period; Park St., Subway & Holy Well Park inundated briefly; ice jam occurred belowhighway bridge at Murray causing inundation of Montgomery Drive area.
March 1979	Heavy rainfall	Marsh between Salmon & North Rivers inundated; Park Street inundated for some time; McClures Brook very high, extent of flooding not known; At Bible Hill, Subway was flooded.
December 1979	Heavy rainfall	Park Street closed because of inundation; the Fundy Trail Mall parking lot partially submerged for period of time.

TABLE 3.1 (cont'd)

Date	Cause	Description/Location
March 1980	Rain, high winds, tides and ice jams	Ice jams occurred at the confluence of the Salmon and North Rivers, at the North River Bridge and on the Salmon River downstream of the Cheese Factory. The approaches to Park Street bridge were inundated for a period of time.
July 1980	Rain	Minor flooding on Queen Street as a result of summer thunderstorms.
January 1981	Mild temperatures, ice jam	Ice jammed in North River downstream of Stewarts Bridge resulted in basement and highway flooding. Damage payments to residents subsequently totalled approx. $50,000.
February 1981	Heavy rain, mild temperatures resulting in series of ice jams	Flooding resulted along North & Salmon Rivers as well as Lepper Brook as a result of ice jams; several roads and streets were blocked; homes damaged and evacuated as well as commercial establishments. Both Fundy Trail Mall and Truro Mall were reported to be threatened.
August 1981	Heavy rain	Overtaxed storm sewers resulted in flooded streets and basements in the Truro area.
February 1982	Mild temperatures resulting in ice jams	Four ice jams were reported to have occurred on North River and one on the Salmon River upstream of Valley but no substantial flooding resulted.
March 1982	Mild temperatures and rain	Resulted in the inundation of Park Street by the Salmon River.
April 1982	Heavy rain	Park Street inundated.
December 1982	Heavy rain and melting snow	Salmon River overflowed banks in the Park Street area.
July 1983	Torrential rain	Record breaking rainfall resulted in localized flooding throughout Truro; Queen St. was reported to be the hardest hit area.

TABLE 3.1 (cont'd)

Date	Cause	Description/Location
January 1984	Rain, mild temperatures, ice jams	Ice jams in the Salmon and North Rivers resulted in flooded basements, roads and farmland.
February 1984	Rain, mild temperatures, melting snow coupled with ice jams remaining from previous week	Resulted in flooding at Marshall's Corner, Park Street and along East Queen St., water levels slightly exceeded those of January.
February 1984	Ice jams, rain	Park Street closed to traffic on the 16th and remained closed until early on the 17th due to inundation. Ice jam on North River posed no threat as dykes had been repaired after previous flood.
March 1984	Rain and ice jams	Ice jams reported near Park St. bridge and at railway bridge on North River. Minor flooding occurred. Park St. closed for a period of time.
February 1985	Ice jam	Ice jam on Farnham Brook resulted in inundation of at least one residence.
January 1986	Heavy rain, melting snow and some ice jams	In Truro area many homes and streets inundated. Ice blocked culvert at North River resulting in flooding of 3 homes and section of Hwy. 311. Another ice jam at confluence of North and Salmon Rivers resulted in flooding at Marshall's Corner, water also spilled into marsh behind Truro Mall. Berry's Furniture Store on Robie St. surrounded by water. Parking lot at Fundy Trail Mall inundated to depth of 0.5 m. Flooding also reported in Park St. area. Farnham Brook overflowed its banks forcing residents of at least 2 homes to evacuate. An ice jam on the Salmon River downstream of Cheese Plant caused significant flooding along Salmon River Road.
March 1986	Mild temperatures, melting snow, heavy rain and ice jams	Minor flooding from several ice jams occurred with ice then moving downstream resulting in significant jam at the confluence of North and Salmon Rivers. Park Street reported to be flooded on both sides of bridge. Hwy. 311 inundated in two sections.

historical data presented in Table 3.1, other information related to the flooding of Park Street was obtained from records maintained by the Nova Scotia Department of Highways. It was found from a review of these records that closure of Park Street has frequently been necessary in the past due to the occurrence of high water and ice over the street on the north side of the bridge near the sewage treatment plant. For example, from 1976 to 1979, twenty-two closures were noted with an average duration of about one day for each closure.

Studies by Maritime Resource Management Service and its predecessor in the early 1970's provide as useful a compilation of historic flood data as existed at that time. A study by Carson of "Ice Jam Effects on Flood Levels at Truro, Nova Scotia", 1978 draws on the M.R.M.S. reports, and generally summarizes relevant flood elevation data. (Relevant details of previous reports are given in Chapter 4.0.)

Reference to Table 3.1 will indicate that there appears to be a void in the data base for the period 1937-1945, which may reflect the fact that flooding was not as newsworthy during the second World War. During this period, a storm in September of 1942 caused extensive flooding along the East River of Pictou and in the Stewiacke Valley. This particular storm deposited about 350 mm of rain at Stellarton. Therefore, one would also have expected flooding in the Truro area during that period.

Generally, heavy rainfall is mentioned as the main factor causing serious flooding (about 90% of the cases) while ice jams and snowmelt are mentioned a significant number of times (40-45% respectively). Tides are mentioned as a factor in flooding for about 15 to 20% of the historical events. In general, flooding is also caused or at least aggravated by man-made structures such as roads, bridges, commercial and residential developments and dykes, etc.

The following list is an attempt to generalize the most frequent locations of occurrence of historical flood damages in the study area (refer also to Figure 3.1 which also identifies the flood prone areas/damage centres according to the following numerical sequence):

1. damage to structures along Robie Street
2. damage to structures along Robie and Juniper Streets
3. residential, commercial and industrial damages (houses, education centre, arena, etc.) in North and Park Street Areas
4. residential damages; upper Onslow Area, including road closures
5. residential/commercial damages; in Marshall's Corner area; including road closures
6. residential/commercial damages; Farnham Road area; including road closures
7. residential/commercial damages; in Riverside/Avon Street area; including road closures (dykes recently built)
8. residential damages; Cottage/Lepper Brook area; dyke recently built)
9. residential/commercial damages along Salmon River Road
10. residential damages, Montgomery Drive Area; most notable damage in 1973; houses relocated; some potential for damage to existing houses
11. residential and commercial damages at North River in vicinity of Stewart Bridge.

Previous studies and surveys document ten significant damage producing floods from 1947 to 1974, and records indicate that there are many earlier events of note. Unfortunately, little useful information for either hydraulic or economic assessment was recorded. For example, it was generally not possible to determine detailed flood profiles along the study area from the descriptive records. Furthermore, it is surprising to note that accurate dollar damage values are not available for floods as recent as the 1974 and 1978 events.

However, it was possible to obtain some reasonably accurate estimates of maximum flood elevations at specific locations for significant damage producing events as summarized in Table 3.2. In summary, in the area from Park Street to the Walker Street bridge, the maximum historical flood level was about 11.7 m. For the area from the Highway 102 crossing up to the North River, the maximum historical flood levels have been about 10.1 m. By comparison, the approximate elevation of the dykes in these areas is roughly 11.7 m and 9.9 m respectively.

3.3 General Description of Flood Damage Factors

The historical review of flood conditions revealed that a combination of natural and man-made factors influence flooding in the study area.

Natural factors such as rainfall and snowmelt contribute to produce high runoff for short time periods on an almost annual basis. The resulting high water levels can be aggravated by ice jams, tidal variations and silt accumulations on the river bottom. Table 3.3 summarizes the natural factors influencing significant flood events, as determined from available records. The natural factors affecting flooding are described in general in Section 3.3.1 and in more detail in subsequent parts of the report.

The development of the floodplain lands adjacent to the river also contributes to the flooding problem. In fact, without development in floodplain lands, economic losses due to natural flooding would be minimal. Man-made factors include fill and related development in the floodplain, occupation of the floodplain and a significant amount of construction of dykes and other hydraulic structures (e.g. bridges, etc.). The man-made factors which influence flooding are described in general in Section 3.3.2.

TABLE 3.2

Estimated Flood Elevations for Selected Significant Historical Events

Location	Elevation (GSC (m)	Date	Comments
Marsh d/s Hwy. 102	9.9 +	1971	Water overtopped dyke at 9.9 m
Robie St./Hwy. 102	9.7	1971	Water 0.6 m over road through underpass
Old Road to Board Landing Bridge	10.0	1952	Water flowing 0.6 m over road
Machine Shop	9.7	1971	Water flowing 0.6 m over floor
Fundy Mall	10.1	1971	Water .75 m over floor
Dyke u/s #102 Bridge south side	10.1	1971	Water flowing over dyke at 9.9 m into river
Juniper Street	9.9	1971	Water flowed east over road at 9.9 m
Lorne/North Streets	10.9 +	1974	Street at 10.9 m covered
Park Street Bridge	10.3	1956	Ice jam just downstream
Park Street, north side of bridge	10.9	1962 1980	Road under 0.6 m of water and ice
Park Street area south side of Salmon River	9.9 +	1962	Flow over dykes onto marsh .15-.30 m deep between Park Street and North River confluence. Dyke elevation was 9.9 m
D/S Stanfield's	11.7	1971	Ice jam at Park Street Bridge, south dyke at 11.74 m, max. water elev. at 11.68 m.
D/S Walker Street bridge	11.6	1952	Piles cracked off, and boiler room flooded .45 m at Stanfield's

TABLE 3.2 (cont'd)

Location	Elevation GSC (m)	Date	Comments
Walker Street bridge	12.2	1952	
Avon Street area	11.1	1952	
Bible Hill, Main St. at Subway	13.1	1947	Water said to be .45 m deep at subway
Queen Street, at CNR Bridge over track	16.0	1935 +1952	Water flowing in cellar door at C. A. Roberts
Queen St. near Dairy	20-22	1935?	Photo of thick ice deposited on street
Montgomery Drive area	24	1973	Ice jam diverted water and ice, causing damage to houses. Houses have since been removed
North River u/s of Railway Bridge	10.8	1962	Road at 10.2 m under 0.60 m of water

TABLE 3.3
Summary of Natural Factors Influencing
Major Flooding Events

Date		Runoff	Ice	Tide	Silt	Area Flooded
1935	Jan.	M	M	N	N	Upper Salmon
1947	May 2	M	N	L	U	Area Wide
1952	Jan. 21 to 24	M	M	L	U	" "
1956	Jan. 6 to 10	M	M	L	M	" "
1957	Jan. 22	M	M	L	M	" "
1962	Apr. 8 to 10	M	N	M	U	" "
1971	Feb. 14 to 15	M	M	L	M	" "
1973	Feb. 3	M	M	N	N	Upper Salmon
1974	Feb. 23	M	M	U	U	Area Wide
1978	Jan. 14 to 16	M	U	N	U	" "
1981		M	M	N	N	Upper North River
1984	Jan.	M	M	N	U	Lower North River
	Feb.	M	M	N	U	" " "

M = Major
L = Minor
N = Nil
U = Unknown

3.3.1 Natural Flood Factors

i) Precipitation and Snowmelt

The Truro area receives about 1100 mm of precipitation yearly, of which over 65% is in the form of rainfall. The remaining precipitation falls in the form of snow from December to March. The precipitation is fairly evenly distributed throughout the year. However, the area is subjected to severe cyclonic storms in the fall and high rainfall and snowmelt in the spring with corresponding increases in river discharge. The maximum recorded 24 hour precipitation at the Nova Scotia Agricultural College was about 133 mm of rain in the month of April. The maximum 24 hour rainfall in the autumn was about 117 mm (Environment Canada, undated).

The majority of damage producing floods in the Truro area are due to rainstorm events. Heavy rain can occur almost any time, but hurricanes during the fall can produce major rainfall events.

Snow melting when combined with heavy rain has also resulted in high river discharge and associated flooding, but snowmelt by itself usually does not cause flooding problems. However, frozen or saturated ground will result in higher rates of runoff from rainfall or snowmelt. Therefore, it can be expected that for an equal amount of precipitation, higher flows would usually be associated with winter-spring events, than during other times of the year.

ii) Ice

About 40% of all flood events occurring in the study area have been aggravated by ice conditions and most of the events of significant magnitude in the study area have involved ice (see Table 3.3). A number of relatively small magnitude discharge events on both the North and the Salmon Rivers would not have caused flooding problems if ice jamming had not been a factor.

Ice jamming may occur in both the tidal floodplain and fresh water sections of the study area as a result of sea ice or river ice breakup. Sea ice formed in the Cobequid Basin can be transported up river by wind and tide water, although most ice related flood problems are likely due to river ice. Ice jams have formed at numerous locations along the length of the river (see Section 4.8). For example, in February 1971, two main ice jams formed, one immediately above Highway 102 and the second at the confluence of the Salmon and North Rivers. Floodwaters overtopped the dykes, flooding the marsh areas to the north of the Salmon River and then overtopping the dykes along the Truro Dykeland Park. Heavy ice pans and ice restrictions at the aboiteau and culverts along McClures Brook reduced the discharge capacity of these structures and increased the rapid buildup of floodwaters on the floodplain.

The North River is confined by dykes downstream from Highway 104, and the Salmon River downstream from the railway bridge (see Section 3.3.2). In both cases, the confined and restricted area is in or near the tidal floodplain, where the river gradient flattens, and the river is less able to transport ice.

In summary, tidal ice forms in a confined outlet; river ice moves from upstream reaches into a confined area; and the river gradient flattens in the same area. This has resulted in the frequent occurrence of ice jams. In addition, ice jams have occurred at a number of other structures and locations in the study area.

iii) Tides

The cyclical nature of tidal water elevation, and the nature of its inward movement, affect both river outlet and aboiteaux outlet discharge conditions. The tidal characteristics are discussed in detail in Sections 4.0 and 7.0. High tides in Truro are characterized by a very rapid rise to high water and a slow ebb. The tidal effect extends up the Salmon River to between the Park Street and Walker Street Bridges and up the North River to just downstream of the Trans Canada Highway Bridge. However, it is not unusual for tidal ice to completely block the estuary downstream of the study area, preventing tides from entering the river.

It is not uncommon for open water tidal events to reach the top of dyke elevation of 9.9 m. Under such a condition, and since any significant fresh water flood events will span several tidal cycles, dyke overtopping and the related possibility of dyke failure could result in flooding.

Less frequent tidal events (e.g. the 1:18.6 year cycle - refer to Section 7.0), combined with barometric and wind conditions could provide damage producing events. The Saxby Tide/Gale of 1869 was such an occurrence. Wind and tide conditions produced extreme water elevations, resulting in widespread damage throughout the Bay of Fundy region.

iv) Sedimentation

The scour and deposition of sand and gravel along the Salmon and North Rivers results in some local conveyance constrictions and deposition of sediment near the mouth of the rivers. Tidal waters also deposit silt in the river system as they retreat, resulting in channel bottom elevation changes in the order of two and a half metres at the Board Landing Bridge. Re-transport of this silt seaward is dependent on fresh water flows of a volume and velocity capable of scouring the channel.

It is likely that the absence of large flows in late fall in some years, has left the channel with significantly less capacity to carry freshwater flood flows in the spring, and further, results in a higher likelihood of ice accumulating and anchoring to the bottom.

Some limited data has been collected over the years which can be used to quantify changes in channel configuration and conveyance due to sedimentation. Figure 3.2 indicates the dynamic nature of the tidal portion of the study area based on field surveys of cross-section changes at the Park Street Bridge.

3.3.2 Man-made Flood Factors

A number of man-made factors influence flood conditions in the study area including dykes, fill and related development, floodplain occupation and gravel operations, etc. These factors are discussed in general terms in the following sections:

i) Dykes

The locations of dykes in the study area are shown on Figure 3.1. Dykes have been modified and upgraded a number of times since their construction first started around the time of the Acadians.

Prior to approximately 1953, the dykes along the study area only provided protection against normal tides. Government programs related to agricultural production provided the means to upgrade the dyking and drainage system on the marsh, through the 1950's. The dyke system was generally upgraded to 9.8 m downstream of the North and Salmon Rivers' confluence. An exception is along the north bank from the McCurdy Brook aboiteau upstream to the C.N.R. bridge on the North River, where the elevation was recently upgraded to 10.1 m and above. This usually provides sufficient protection against maximum annual tides.

On the south side, a length of approximately 300 m immediately downstream of the Park Street bridge was generally raised to 10.65 m following substantial overtopping in that area due to ice jamming in 1956.

At the downstream end of the North River/Farnham Brook marsh area there is a 60 m long section of dyke at an elevation of approximately 9.4 m. This directs flow into the North River near its confluence with the Salmon River.

Upstream of Park Street, there is no dyking on the north side. The dyke elevation on the south side rises from 10.6 m at the Park Street bridge to an elevation of 12.5 m near Stanfield's plant. For an additional 300 m upstream, a retaining wall behind Stanfields has a top elevation in the order of 12.2 m. The area between the Walker Street bridge and the C.N.R. bridge, south side, was dyked in 1953 to an elevation of 14.3 metres.

In 1984-85 a new berm was constructed for a distance of about 1500 m upstream of Walker Street along the south bank of the Salmon River for the purposes of controlling ice movement and damage. A short length of berm was also subsequently constructed on the north side of the river from the Walker Street bridge to Holywell Park.

On the North River, dykes extend upstream from the confluence to the vicinity of the Trans Canada Highway. A portion of these dykes was upgraded to an elevation of approximately 10.4 m in 1978.

Several aboiteaux are built into the dykes in the study area. The flap gates located on the downstream side of each aboiteau provide for the release of flow from the dyked areas, while preventing flow from the river to areas behind the dykes, depending on the tidal cycle and water levels in the river. The aboiteaux in the study area are listed in Table 5.4. The reference numbers on Table 5.4 are used to locate the structures on Figure 5.1.

ii) Fill and Related Development

In addition to dykes, fill for construction of hydraulic structures, etc. has been placed at a number of locations in the study area. A list of structures crossing the Salmon and North Rivers in the study area is given in Tables 5.2 and 5.3. The reference numbers referred to in Tables 5.2 and 5.3 are used to locate the structures on Figure 5.1.

The C.N.R. and D.A.R. (Dominion Atlantic Railway) lines parallel and/or traverse the floodplain at several locations in the study area. These rail lines may impede the movement of floodwaters. In some instances, they may provide flood relief by acting as a secondary dyke.

The location of a number of highways and roads are also indicated on Figures 3.1 and 5.1. With the construction of Highway 102 came the associated problems of conveying floodwaters from Truro Dykeland Park. This highway, considered a barrier to the passage of water except for the outlet via the Princeport Highway, resulted in the construction of the viaduct to the north of Robie Street. Highway 102 and related structures cross the floodplain at an elevation of 13 metres, approximately 4 to 5 metres above the floodplain. The rerouting and consolidation of McClures Brook and other drainage into one system with an outlet through an aboiteau structure, just downstream of Hwy. 102, eliminated the Moose Creek aboiteau. McClures Brook formerly passed under Highway 102 through a 3.96 m culvert. This was replaced by an 18 m bridge structure in 1982. Robie Street underpasses Highway 102, and this structure has carried flood waters in the 1971, 1974 and 1978 floods. In 1983, Marshland Drive was constructed by the Town along the south edge of the floodplain between Park Street and Robie Street at an elevation of 9.0 to 9.5 m.

The subway under the C.N.R. near the Walker Street bridge was previously inundated during major floods in the Truro area. This inundation has varied from a few centimetres to one and a half metres. However, the recent construction of a dyke between Walker Street Bridge and Holywell Park has reduced the possibility of flooding for all events with flood water elevations under about 14.0 m.

Dykes, bridges, and fill for transportation and other structures may limit the passage available for flood flows, and ice carried by such flows has a greater likelihood of jamming and thus further reducing passage at such points.

Substantial commercial development along Robie Street, Juniper Street, and the Lower Truro Road, and particularly the fill associated with it, has altered the floodplain. Future flood flow and storage will no doubt be affected by these factors.

In the general area of the Park Street bridge, filling associated with the construction of a sewage treatment plant took place in 1971. The old bridge also tended to create ice jamming and backwater at this location until the construction of a new structure in 1984 which has considerably improved the hydraulic characteristics at this location. The approach road to the bridge was kept to the existing grade at the northeast side to allow access of floodwater and ice to the natural storage areas in Farnham Brook Marsh.

Some floodplain users have also generally raised some portions of flood prone lands by dumping fill in the floodplain. For example, upstream from Park Street on the north bank of the river, Domtar Limited raised the floodplain by filling during the early 1970's.

iii) Gravel Removal

Gravel removal operations have been undertaken at various times in the past at several locations in the study area, although the quantities removed are not known. In addition to the environmental consequences of gravel removal, the operation has created several areas where the resulting channel and floodplain is irregular in cross-section, with an associated potential increase in flood hazard due to ice jams. All future gravel removal operations must be approved by the Ministers of the Nova Scotia Department of the Environment and Fisheries and Oceans Canada.

A review of records maintained between 1967 to 1977 by the Nova Scotia Department of the Environment indicated that the removal of gravel along the Salmon River occurred primarily upstream of Truro, although gravel has been removed occasionally in the Park Street and Walker Street areas. Along the North River, gravel has been removed from time to time in the area of the Trans Canada Highway Bridge and less frequently upstream of North River.

iv) Channelization

There has been considerable activity over the years to enlarge the capacity of the Salmon and North River Channels. The most notable examples of this were:

1. the North River from Marshall's Corner to Highway 104 in the 1960's
2. the Salmon River in the vicinity of the Murray Siding Bridge in the 1970's
3. the North River upstream and downstream of Stewart's Bridge in the 1980's

It has been observed that this type of project may aggravate flooding downstream, particularly during ice jam events. The physical characteristics of these and other projects in place prior to 1985 have been incorporated into this study.

v) Floodplain Occupation

The floodplain has been used in the study area since the 1600's. Initially, dykes were constructed to prevent inundation of crops by normal tides. However, the use of the floodplain lands has diversified and greatly intensified over time. Confining normal tidal events with dykes at the river's edge proved to be a practical and economical undertaking; while attempting to confine abnormal tidal events, high spring freshwater events, water levels produced by ice jams, or various combinations of these factors, has proven to be impractical, if not impossible. In spite of this, inappropriate development of the floodplain has continued, with the assumption that the dyking system affords protection.

The dry viaduct under Hwy. 102 just south of the Salmon River has carried flood waters estimated to be 1.8 metres deep. However, the Pallisers Road and the remnant dyking associated with the defunct Moose Creek aboiteau place limitations on the capacity of this structure.

The commercial developments along Robie Street, Juniper Street and Lower Truro Road were previously discussed in regard to obstruction to flow due to associated fill in the floodplain.

The substantial institutional development in the Lorne and North Streets area, although not constructed on fill, may also affect flood water storage, and may be affected by flood water damage.

Marshland Drive, running from Park Street to Robie Street may also have some effect on flood water storage by preventing access to some areas of the floodplain. This road now forms the southern boundary of the floodplain available for storage of flood waters.

Any other additional uncontrolled uses of the floodplain in the future would also likely have additional adverse effects in regard to flooding characteristics.

3.4 Existing and Potential Damage Magnitude

3.4.1 Existing Damage

Most of the damage centres depicted on Figure 3.1 remain susceptible to flood damages. The magnitude of potential flood damage in the 1980's would be greater than historical flood damages due simply to the increased value and amount of development that would be affected by flooding. Recent recognition of the flood risk is evidenced by a number of recent developments which have been constructed at high elevations on fill. In spite of these isolated attempts at floodproofing, there would be significant damage in the form of lost revenue, loss of access, and the possibility that the floodproofing elevation chosen would be exceeded.

Both tangible and intangible losses must be considered when determining the magnitude of flood damage. Direct water damage and water borne ice and debris damage to structures and contents can be tabulated subsequent to an event, and methods are available to provide direct damage estimates for assumed flood elevations. Other indirect damages are not so easily calculated, but must include consideration of the following factors:

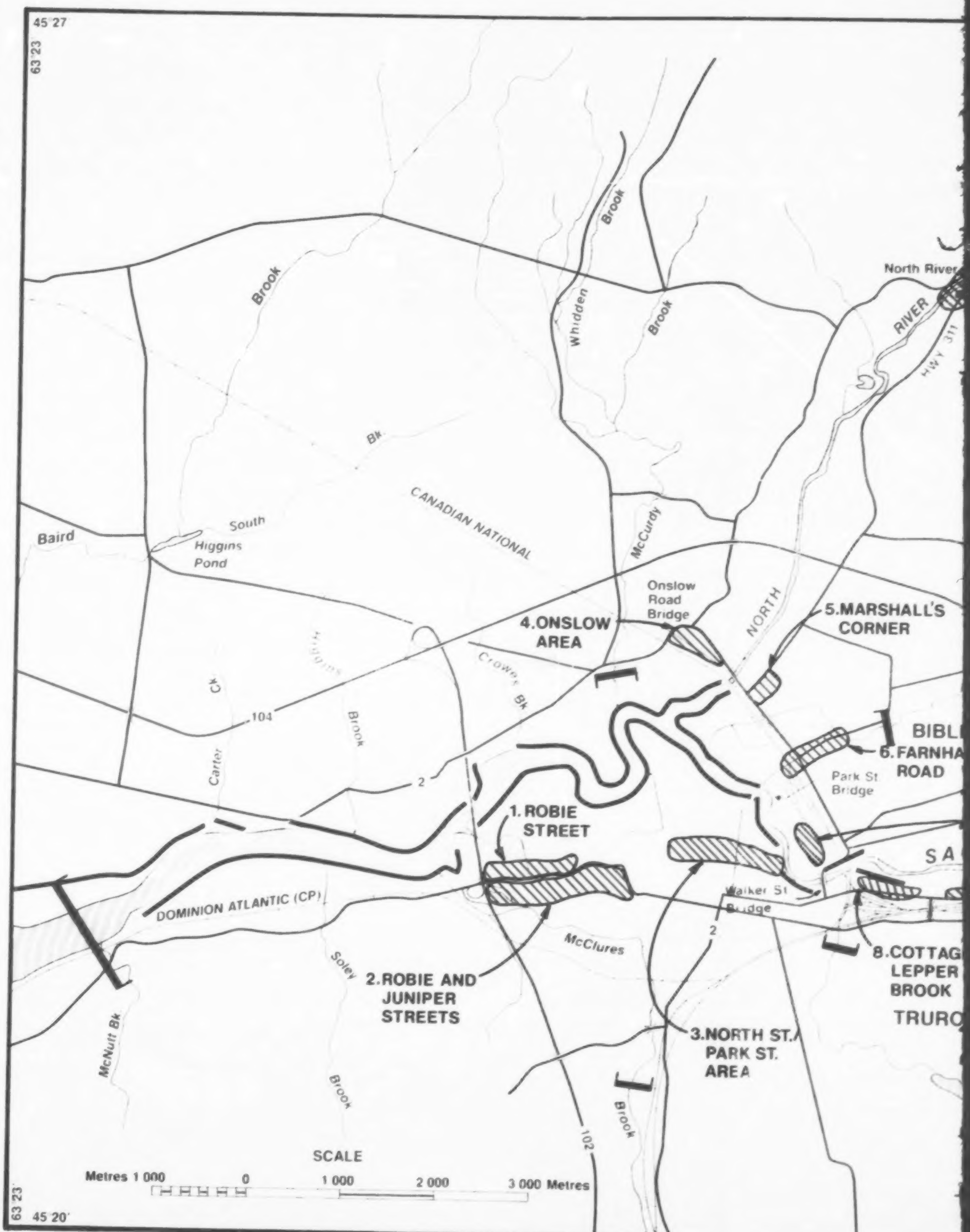
1. social and commercial disruption - loss of personal income, destruction of records and personal artifacts, loss of commercial revenue, loss of tax assessment
2. safety - isolation from essential services - fire, hospital, abnormal risk to life incurred in combating flood waters, performing rescue, or undertaking salvage
3. public - costs of emergency and rescue works, restoration of services and streets, equipment, staff; the provision of works to attempt to reduce or contain flood waters for future events.

Dollar estimates of damage do not appear in the records for most flood events. However, there are two instances of direct damage related public costs. The first is the relocation of several houses from Montgomery Drive subsequent to the Salmon River ice jam event of 1973 and the second was compensation for flood damages to various residents affected by the North River ice jam event of 1981 at Stewart Bridge. The total payment for the latter event was about $50,000. Very significant additional costs have also been incurred in providing post flood solutions in the form of dyking, river dredging, bridge and culvert improvement.

3.4.2 Future Damage

There is little doubt that when future floods reach elevations experienced several times in the past 40 years, greater levels of damage will occur than in the past. The intensive development of West Robie Street in the years since Hwy. 102 was built will inevitably result in increased damage. Recent development elsewhere throughout the Salmon and North River floodplains will suffer damage as well.

There is also little doubt that the potential exists for higher flood elevations compared to available records of historical flood levels. Prior studies have indicated that an elevation of 10.65 m could occur in the lower tidal floodplain, whereas the highest elevation on record for this area is approximately 10.1 m. This has led to the use of a flood proofing elevation requirement of 10.65 m by the Town of Truro in recent years. However, to date, there is no such land use control in the county areas (outside the town limits) other than in the Village of Bible Hill.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CORNER
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104
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STREETS
8.COTTAG
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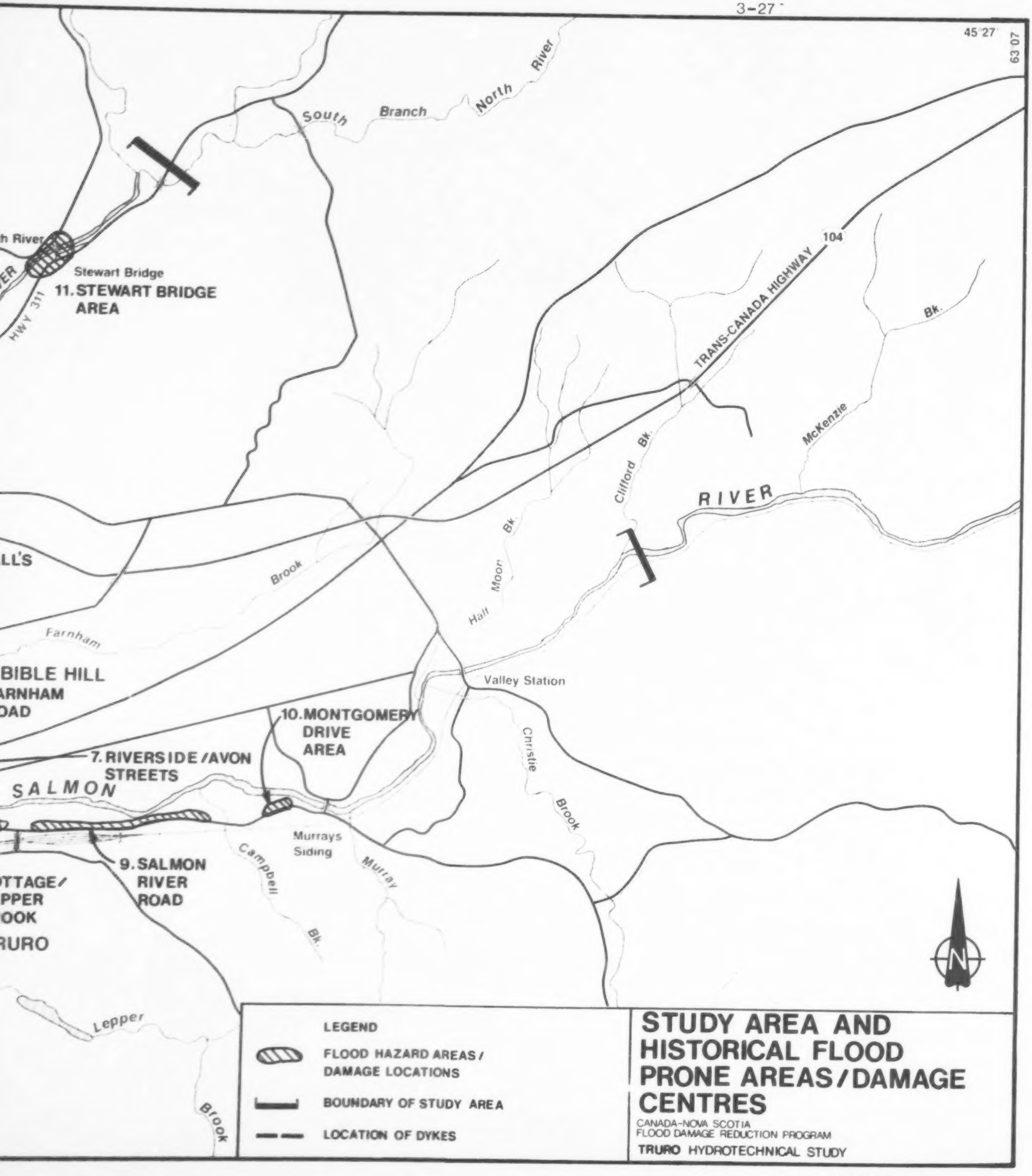
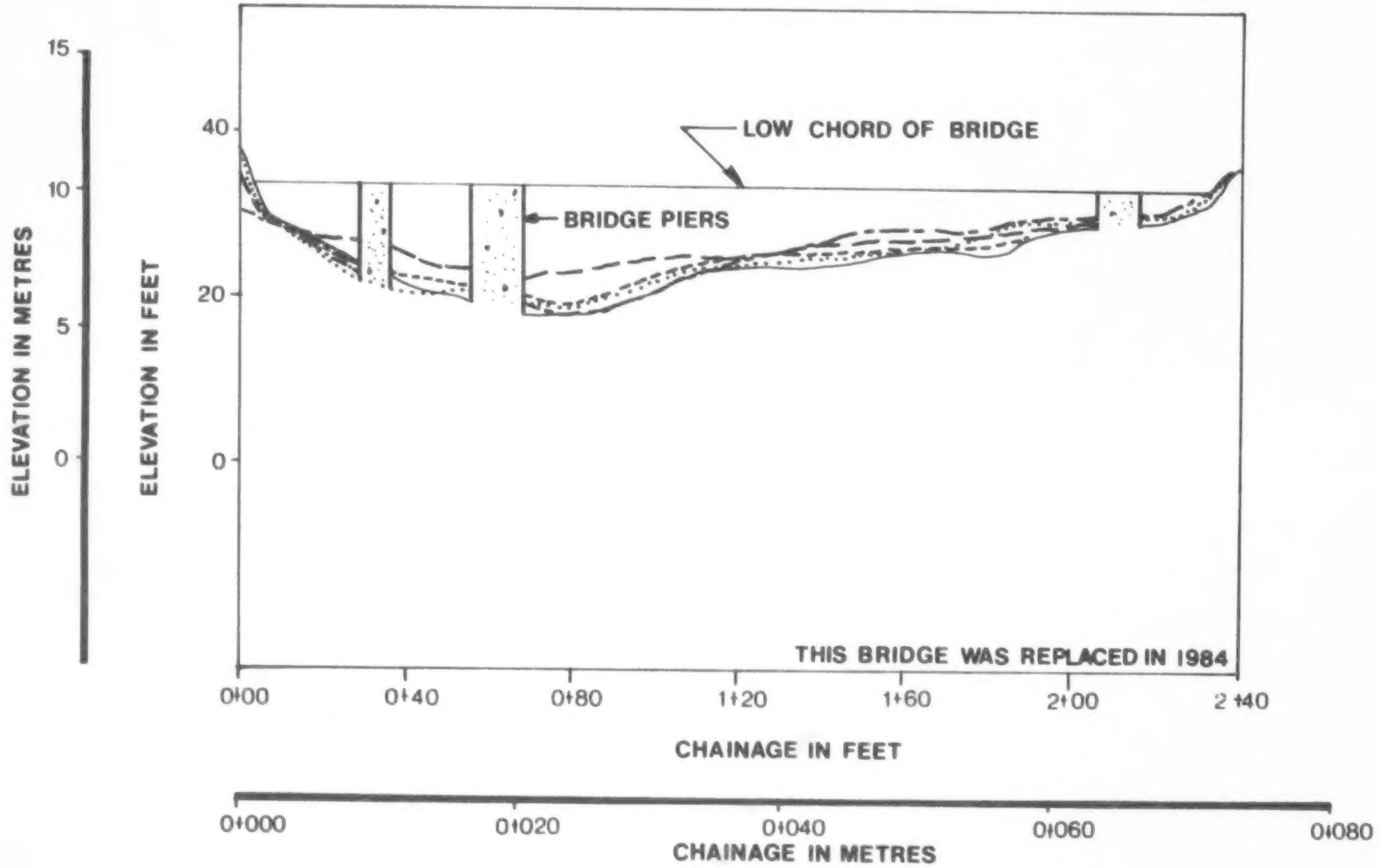

45°27'
63°07'
North River
South Branch North River
Stewart Bridge
11. STEWART BRIDGE AREA
HWY 311
TRANS-CANADA HIGHWAY 104
Bk.
McKenzie
Clifford Bk.
RIVER
Half Moon Bk.
Brook
Farnham
BIBLE HILL
ARNHAM
OAD
Valley Station
10. MONTGOMERY DRIVE AREA
7. RIVERSIDE/AVON STREETS
SALMON
Christie Brook
Murrays Siding
Campbell Bk.
Murray
9. SALMON RIVER ROAD
OTTAGE/
PPER
OOK
RURO
Lepper Brook
N
LEGEND
FLOOD HAZARD AREAS/ DAMAGE LOCATIONS
BOUNDARY OF STUDY AREA
LOCATION OF DYKES

STUDY AREA AND HISTORICAL FLOOD PRONE AREAS/DAMAGE CENTRES

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

TRURO HYDROTECHNICAL STUDY

FIGURE 3.1



SOURCE: (ENVIRONMENT CANADA, 1978)

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

**CROSS SECTIONS
SALMON RIVER AT
PARK ST. BRIDGE**

FIGURE 3.2

4.0 PREVIOUS STUDIES

4.1 Introduction

Several studies of flooding problems in the Truro area have been undertaken primarily in an attempt to develop a method for reducing the severity of flooding. This section provides a brief outline of the more pertinent reports and a summary of the corresponding recommendations. Since this section is a review of previous investigations, the findings presented are not necessarily consistent with current findings and conclusions expressed in other sections of this report. Unless specifically noted in the subsequent text, the remedial measures suggested by various investigators were not undertaken.

4.2 Bell-Dawson - 1917

One of the earliest and most comprehensive reports on the tidal regime of the Cobequid Bay was compiled by W. Bell-Dawson and published by the Department of Naval Service in 1917. His task was to make the navigation of the numerous small steamers in the upper part of the Bay safer. The steamers had to call at the harbours at high water and had to leave according to the stage of the tide, as most of the wharves ran dry at about half tide. According to his report, schooners were accommodated by lying on a bench of mattresses against the wharves while the tide was down. His early attempts to measure the tides at Truro failed due to the difficulty of access. The farthest easterly point in the Bay where he could reach the tide was at Burntcoat Head.

With respect to the objectives of this study, Bell-Dawson made two major contributions. Firstly, he determined that the tide at Burntcoat Head was almost perfectly sinusoidal, and that it can be determined with some confidence based on Saint John tidal data. Secondly, he determined the elevation of the much discussed "Saxby Gale" which occurred on October 4, 1869, at a time of extreme high tides. Based on his data, the storm surge associated with this event has been estimated at 0.93 m at Noel River near Burntcoat Head.

4.3 Cox, O.S. - 1938

The first known report outlining a flood control program for the Truro area was written in January 1938 by Mr. O. S. Cox, who was District Engineer at that time for the Federal Department of Public Works (Cox, O.S., 1938).

The report suggested two methods to control flooding along the Salmon River between Murray Siding and the railway bridge in Truro:

1. "By a system of artificial reservoirs constructed on the reaches of the river..."
2. "By a system of channel rectifications between the Vernon Bridge (Murray Siding) and the railway bridge in Truro..."

There was no elaboration on the first proposal, as it was considered outside the scope of the survey work done, and the information essential to deal with the suggestion was not available. However, more detail was provided on the proposed channel rectifications. The report contained a map of the Salmon River from Murray Siding to the railway bridge showing a number of surveyed cross-sections of the floodplain.

The proposal suggested the construction of a straight channel between the two bridges, and the installation of cribwork dams where the new channel intersected the existing river channel. The proposal also called for the construction of 3224 lineal metres of dyke embankment and the general levelling of the valley floor.

In a review of Mr. Cox's report, Mr. C. A. Menard, also with the Department of Public Works, recommended the collection of more field information before undertaking any channelization. He suggested that gauging sites should be established along the river reach at each break in the water level, and at each material change in the cross-section of the river. Water surface elevations should be taken at each gauge site immediately before the beginning of the freshet, during ice movement at peak flows, and at various stages of the river. It was also recommended that flow measurements be made each time the water level was recorded.

None of the recommendations made by Mr. Cox or Mr. Menard were acted upon.

Three flood levels were also noted on the plan. At the pedestrian subway in Bible Hill, a freshet elevation of 13.66 m was given, but no date was indicated. Approximately 1067 m upstream from the railway bridge a freshet elevation of 15.95 m for the spring of 1935 was noted. A flood elevation of 18.6 m for January 1935 was also shown at a point approximately 1829 m upstream from the railway bridge. There was no indication that any of the three high water levels occurred during the same flood, but other reports indicate that widespread flooding took place throughout Nova Scotia between January 10-15, 1935 (see Table 3.1).

4.4 Byers, J.W. (unpublished) - 1947 and 1952

Mr. J. W. Byers made extensive notes concerning the flood of April 30 to May 3, 1947, during which flood waters from the Salmon River entered the Truro marsh downstream from Stanfield's plant. At that time, the dyke existed above Park Street on the south bank. The high water mark at Stanfield's was reported to be approximately 12.2 m geodetic. Later in 1947, it was reported that about 274 m of the river below Stanfield's was widened. The gravel was bulldozed up from the river bed into a dyke.

Notes were also made by Mr. Byers concerning high water levels during the flood of January 21-24, 1952. Comments provided by one resident indicate that the peak water levels occurred about 8:00 p.m., January 23, and the water level started to drop about 3:00

a.m. on the 24th. A number of water elevations were taken during the flood, unfortunately, it seems that only a few were converted to geodetic. Those that are available are summarized as follows:

Date	Time	Location	Water Level (G.S.C.)
21 Jan. 1952	3:30 p.m.	Walker St. Bridge	12.23 m
21 Jan.	4:55 p.m.	Stanfield's Plant	11.56 m
21 Jan.	-	Park St. Bridge	9.63 m
24 Jan. 1952	3:35 p.m.	Walker St. Bridge	11.71 m
24 Jan.	4:05 p.m.	Park St. Bridge	9.18 m
24 Jan.	-	Stanfield's Plant	11.10 m

4.5 Atlantic Tidal Power Programming Board - 1969

The Atlantic Tidal Power Programming Board conducted extensive studies of the tidal regime in the Bay of Fundy, culminating in 1969 with the publication of its final report (Atlantic Tidal Power Programming Board, 1969).

Of particular relevance to this study, the Board considered the significance of atmospheric disturbances on water levels and improved the level of understanding of the astronomic tidal regime in the Bay through the establishment of additional temporary tidal gauges at various locations.

The Board concluded that a hurricane surge (atmospheric pressure drop plus wind set-up) of 1.72 m is possible at Cape Split. The Board also determined that a wind set-up of 0.6 m was possible at high tide at Burntcoat Head (Site 8.1) in the Minas Basin, and that the frequency of this event would be approximately 1:20 years.

4.6 DREE - Flood Hazard, 1971

As the result of severe flooding in the Truro area on February 14, 1971, two reports were prepared by the Engineering Branch, Department of Regional Economic Expansion in Amherst.

This first report, dated February, 1971 gives a brief outline of the geology, meteorology, hydrology, tides and the past history of flooding in the Truro area. The main factors affecting the flood hazard were identified as excessive discharge, high tides, ice jamming, change of river gradients near the town, infilling of the floodplain, dyke alignment and sedimentation of the river bed.

The report made three recommendations for the alleviation of flooding in the Truro area:

1. construction of a tidal causeway dam
2. development of floodplain zoning regulations
3. further investigation and study of the effects of headwater retardation dams and channel improvements be undertaken.

No data was provided to support the recommendations and no cost estimates were made.

4.7 DREE - Preliminary Report Flood Alleviation, 1971

This report, dated July 1971, expanded somewhat on the subject matter of the previous report, dated February 1971. It was pointed out that the streams have an average slope of 0.75 percent to 1.5 percent, and this steep gradient changes abruptly in the estuary.

The section of the Salmon River between Murray Siding bridge and the railway bridge was found to have a gradient of 0.2 percent. From the railway bridge to Walker Street, the gradient was determined to be 1.0 percent. Below the Walker Street bridge, the transition between fresh water and salt water occurs, and the gradient decreased to 0.15 percent (varying between 0.07 to 0.36 percent). The river bed in this section was found to be susceptible to tidal silting and it was noted that the bottom elevation could vary between geodetic elevation 5.70 m and 7.35 m.

The report states that a transitional section exists in a tidal river in which the tidal regime is dominant most of the year, but is replaced by a freshwater regime during periods of high runoff. The transitional section, therefore, undergoes seasonal changes in shape and dimensions as the dominant flows change. At times of low river discharge, the tidal regime attempts to alter the channel by filling it with silt to a gradient which will be balanced by the outgoing tidal flow. During periods of high freshwater flows, the channel is reshaped to accommodate the higher flood flows.

The report offered the following flood alleviation proposals to deal with the high freshwater flows, high tides, tidal siltation and ice jamming problems:

1. construction of tidal dam. This was considered the first and major measure to alleviate flooding
2. dyke improvements and dyke levelling. The report proposed raising approximately 4268 m of dyke on the Truro Dykeland Park marsh, and levelling approximately 1525 m of dyke on the Lower Truro marsh
3. viaduct improvements. The dry land viaduct through Highway 102 is partially ineffective in carrying off extreme flood waters because of obstacles upstream and downstream from the structure. No details were given in regard to the proposed changes to the approach and exit of the viaduct.
4. improvements to the McClures Brook Aboiteau. It was proposed that the aboiteau be removed and replaced by a protected outlet structure designed to permit free discharge from the brook. Also, it was recommended that a dyke be constructed from the west end of Prince Street to Highway 102, at an

elevation high enough to contain the flood waters within the floodplain area.

5. additional relief could be obtained by adding a multiplate culvert through Highway 102 and the Princeport Road to accommodate McClures Brook. (The culvert under Highway 102 was subsequently replaced by a bridge during the summer of 1982.)
6. construction of four headwater retardation dams. These were considered supplementary protective measures that would mostly benefit upstream areas.

4.8 Cullen, A.J. and T.M. Humes - 1974

During 1973 and 1974 a study of the effects of channelization on the North River above the Trans Canada Highway bridge (Highway 104) was conducted by Fisheries and Environment Canada (Cullen, A.J. and T.M. Humes, 1974). The initial field data was obtained just prior to a major reshaping of the North River Channel from the Trans Canada Highway to a point some 448 m upstream.

The study found that after reshaping, the channel was wider and the slope greater than that necessary for a stable channel. The conclusions and recommendations of the study are listed as follows:

1. "The regime analysis of this reach indicates that the stream will continue to deposit gravel until it has obtained a stable width, depth and slope."
2. "A serious scour problem is occurring at the centre pier of the Trans Canada Highway bridge and protective measures should be taken to prevent further damage."
3. "If channelization projects are to be carried out with the purpose of improving flow through a reach and not simply for commercial gravel removal, then care should be taken to design a stable channel. If commercial gravel removal is to be allowed by the Provincial Regulatory Agencies, then specialist consulting should be obtained to prevent adverse

effects on existing structures and the aquatic environment of the stream."

4.9 MRMS - Central Colchester Area Flood Study, 1974

In 1974 the Maritime Resource Management Service was contracted by the Department of Municipal Affairs and the Joint Planning Advisory Committee, Municipality of the County of Colchester to undertake a study of the floodplains of the Salmon and North Rivers in order to aid in devising a management plan for these lands. Further advice was requested on:

1. the suitability of constructing a road (either high or low level) from Park Street near the bridge west to join Robie Street just east of the cemetery near the Christian Reformed Church in the Town of Truro
2. the suitability of constructing a road joining the above road at Park Street and extending south to pass through Queen and Prince Street to provide access from the south side of the Town of Truro
3. the drainage adjustments that might be required should Highway Route 102 be twinned across the floodplain
4. the suitability of locating a campground on the floodplain near the second road proposed above.

The report reviewed the history of flooding, factors contributing to flooding, methods of alleviating problems and included a discussion on flood damage. The report also stressed control over use of the floodplain, and provided several recommendations on floodplain development.

The construction of a high level roadway across the floodplain near the present Park Street was recommended, together with a trestle section about 183 m long to pass flood flows. It was also recommended that the proposed road running west from Park Street should be kept as close as possible to the back of the existing developed areas, and have a finished grade at elevation 10.67 m. This level of road should be maintained to its junction with Highway 2 (now Highway 102).

The report recommended that the second proposed road from Park Street to Queen and Prince Streets should also be at a high level to protect the area west of Lorne Street against overbank flooding from the Salmon River. The report also suggested that twinning Highway 102 would increase the flooding problems adjacent to McClures Brook.

The conclusions and recommendations of this report listed eleven activities that should be considered in a flood management program. Some of the activities of interest to the present investigation are as follows:

1. "The preparation of flood risk maps."
2. "Flood forecasting."
3. "Consultation with EMO and/or other agencies on flood fighting and evacuation measures."
4. "The consideration of the feasibility and the implementation of flood control measures."
5. "Recommendations for minimum protection levels and establishment of suggested encroachment limits."
6. "Promotion of the need for the merits of floodplain planning and regulation and adoption of land use controls."
7. "Provision on request, of information of flood risks at specific locations."
8. "Estimation of flood damage potential."

There were also eleven recommendations outlined in this report. Some of the more relevant ones for this present study are as follows:

1. "That detailed flood line mapping be undertaken in areas of high damage potential, and that information on flood damage

potential be widely distributed."

2. "That flood hazards be considered in land use planning for areas encompassing floodplains."
3. "That consideration be given by government to the enactment of a floodway encroachment line statute and its subsequent enforcement."
4. "That local government be encouraged to regulate floodplain occupancy through measures such as zoning and subdivision regulations."
5. "That a program be initiated to provide information on and encourage the use of floodproofing measures."
6. "That flood control be considered in the planning, design and construction of headwater retardation dams in the watershed."
7. "That an integral part of any flood alleviation program be the construction of a tidal causeway dam in the estuary."

4.10 Desplanque, C. - Tides in the Cobequid Bay and Salmon River Estuary, 1977

Mr. C. D. Desplanque of the Maritime Resource Management Service conducted a study of tides in Cobequid Bay and the Salmon River Estuary in 1977. Based on an analysis between tides at Saint John and the data recorded at the Old Board Landing Bridge site, tides at Truro can be predicted to within ±0.45 m. The relationship developed is shown on Figure 4.2.

The report also considered the effects of meteorological disturbances on water levels in the Bay of Fundy. It was considered that the effects of atmospheric pressure on water levels could be as high as 0.5 m and the effects of wind set-up could be as high as 1.6 m, leading to a cumulative storm surge of as much as 2.1 m. The report goes on to say that such an event would be rare, and the probability of such an occurrence with extreme high tide would be even less. It was also pointed out that while wave heights in excess of 0.6 m are theoretically possible along the Salmon River Estuary, they would be extremely rare.

The report concludes that, "it is theoretically possible that high salt water levels in the Salmon River can reach elevations in the order of 10.98 m above Geodetic Datum. The frequency of such an event cannot be determined yet, but it is most likely less than once in 100 years. With such length of statistical recurrence intervals, the fact of submergence of the land mass cannot be neglected."

The overall conclusions of the report indicate that the lack of a tidal station in the Cobequid Bay or in the Salmon River estuary makes the prediction of the frequencies and levels that sea water will reach a difficult and uncertain undertaking. It is pointed out that although the relationship between the Saint John tides and the tides in the Salmon River estuary could be approximated by a single conversion curve, one has to keep in mind that deviations from this curve can amount to ±0.8 m. It was concluded that no analysis is presently available which could explain this wide spread of possible elevations.

4.11 Carson, R. - Ice Jam Effect on Flood Levels at Truro, 1978

In 1978 Mr. R. Carson, at the Canada Centre for Inland Waters, Environment Canada, in Burlington, undertook an investigation of the effect of ice jams on flood levels. To undertake this study, flood water elevations were estimated from high water marks reported in previous studies. Representative levels were determined for six areas: 1) Truro Dykeland Park, 2) Onslow/North River Marsh, 3) North River/Farnham Brook Marsh, 4) Lower Truro Marsh, 5) McClures Brook and 6) Walker Street Bridge area. In order to compare flood levels from different events, the floods were also subdivided into floods with and without ice jams. The results of the analysis are summarized in Figure 4.1.

It was found that the natural complexity of the floodplain, compounded by encroachment and numerous alterations on the floodplain, and the lack of quantitative data made physical analysis impossible. However, as expected, it was found that ice jams form in river bends and gravel bar areas of the river (see Figure 4.1). It was found that the combined effects of dykes and bridges in the study area hold back the jammed ice so the jam cannot break, but that the bridges do not initiate the jams. The only evidence of man-made factors influencing ice jams is the repeated occurrence of jams in the Salmon River near the Cheese Factory and Montgomery Drive following dredging of the river upstream that apparently resulted in more ice moving downstream to a chronic jam location.

A statistical comparison of flood levels with and without ice jams revealed no significant difference between the two conditions. The yearly maximum flood levels attained in each flood area (comprised of both ice jam levels and open water floods) were analysed using available flood records from 1900 to 1975. The records were considered complete and any year without a recorded event was assumed to have had flood elevations below the lowest flood level recorded. The normal distribution was used and the fit was estimated visually, owing to the incomplete record and rather imprecise data. The resulting 1:20 and 1:100 year flood estimates are shown on Figure 4.1.

The report also refers to work by H. R. Kivisild which establishes the critical Froude Numbers above which ice will jam to be 0.08. The Froude Numbers were calculated for different locations, using the available cross-sectional data and discharge from regional flood frequency analysis. The Froude Numbers were found to be much greater than the critical value for ice jamming, even for annual flood discharge values. To reduce the Froude Number to a value below that critical for ice jamming would require a permanent increase in channel depth along the entire river channel. The report also stated that the effects of ice jams are increased by the closeness of the dykes to the river channel. The following are the conclusions from this study:

1. "From the available data, the statistical analysis of floods with and without ice jams leads to the conclusion that ice jamming has no significant effect on flood levels attained ....."
2. "The 100 year flood is clearly not much higher than the 20 year flood because the floodplain requires large floodwater volumes to bring about a small change in elevation. Therefore, all significant floods tend to reach about the same elevation.
3. The analysis also indicates that flood levels are strongly influenced by the construction of elevated roadways in the floodplain. Additional structures or dykes could easily worsen the flood levels. Estimates of the ice jamming propensities of the Salmon and North Rivers shows that conditions for jamming are nearly always present given the breakup of the ice sheet."

4.12 Water Planning and Management Branch - Environment Canada - 1978

In 1978 the Water Planning and Management Branch, Inland Waters Directorate, developed a number of equations relating water levels at Truro to recorded tides at Saint John. The equations were based on regression analysis between recorded data at the Board Landing Bridge and the tidal gauge at Saint John during coincident periods of record in 1971 and 1972 (Inland Waters Directorate, 1978). The appropriate relationship is shown graphically on Figure 4.3. The scatter in the data at high stage was approximately ±0.3 m.

4.13 MacLaren Atlantic Limited - Regional Flood Frequency Analysis, 1980

In 1978 the Government of Canada and the Province of Nova Scotia entered into a General Agreement respecting Flood Damage Reduction, and subagreements respecting flood risk mapping and remedial studies. One objective of the flood Risk Mapping Agreement was to prepare maps for flood prone locations listed in Schedule A of the agreement. The maps are to show the area that would be inundated by the 1:20 year and 1:100 year recurrence interval flood flows. Most of the flood prone locations lacked the hydrometric data required to fulfill this objective. A regional analysis was, therefore, undertaken in order to estimate flood flows at ungauged locations, or at locations with inadequate hydrometric data.

Hydrometric stations with a period of record greater than ten years were used. However, several stations were omitted due to the effects of reservoir regulation on peak flows. The annual maximum daily mean flow data at relevant hydrometric stations was fitted to four theoretical frequency distributions and on the basis of statistical comparisons, the Three Parameter Log Normal Distribution was selected for use in this study.

On the basis of an assessment of watershed climatic and physiographic characteristics, and also using statistical tests, the mainland portion of Nova Scotia was considered to be a homogeneous region with respect to flood discharge characteristics. A regional coefficient of skew was estimated and the Three Parameter Log Normal Distribution was applied to obtain single station estimates for the 1:2, 1:20 and 1:100 year flood peaks.

Linear regression techniques were then utilized to develop functional prediction equations for peak daily mean discharges for 1:2, 1:20 and 1:100 year recurrence interval floods. A similar relationship for estimating the ratio of instantaneous discharges to daily mean discharges was also developed. The study determined that the statistically important physiographic and climatic parameters were area of the watershed, index of mean annual runoff, index of median depth of snow on the ground at the end of March, drainage density factor, and a location factor expressed as degrees longitude.

Using the equations developed in this study, the estimated 1:2 year, 1:20 year and 1:100 year instantaneous flood peaks and the daily mean flood peaks were calculated for the flood prone areas listed in the mapping agreement. The instantaneous flood peaks for several locations in the Truro area are summarized below:

Location	Instantaneous Flood Peaks (m^3/s) 1:2	1:20	1:100
Salmon River at Murray	131.5	228.7	314.4
Salmon River between Farnham Brook and North River	147.7	249.9	341.4
Salmon River at Route 102	209.4	339.4	459.8
North River at North River	81.1	141.3	194.4
North River at Salmon River conflu.	85.6	145.3	198.8
McClures Brook at Salmon River	40.4	59.4	78.7
Lepper Brook at Salmon River	19.8	32.3	43.7
Farnham Brook at Salmon River	20.6	34.9	47.7

4.14 Dillon-Horner - 1980

During 1980, the firms of M. M. Dillon Ltd. and W. N. Horner and Associates Ltd. acting in joint venture, compiled, collected and interpreted tidal data for the Bay of Fundy. These were presented in an unpublished report (Dillon-Horner, 1980). Tidal data collected during the course of the Dillon-Horner investigation is presented in Chapter 5.0. The independent interpretations carried out by Dillon-Horner are summarized in the following sections.

1) Astronomical Tidal Regime

Based on a statistical analysis of extreme tides, as recorded at Saint John during a representative eighteen year period, 1947-1964, it was concluded that:

1. "1 in 100 high tides (7 times a year) will have an elevation of 8.56 m (Chart Datum) or higher"

2. "1 in 1,000 high tides (once every 1.3 years) will have an elevation of 8.75 m (Chart Datum) or higher"
3. "1 in 10,000 high tides (once every 13 years) will have an elevation of 8.99 m (Chart Datum) or higher."

The report suggested that tide magnitude at Truro (Old Board Landing Bridge location) could be predicted based on regression analyses performed by the Water Planning and Management Branch (Inland Waters Directorate, 1978). Based on this, tidal elevations (Geodetic Survey of Canada Datum) at Truro corresponding to those presented above at Saint John were reported as follows:

1. "1 in 100 tides = 9.14 m"
2. "1 in 1,000 tides = 9.51 m"
3. "1 in 10,000 tides = 9.91 m"

An independent analysis of the tidal characteristics of the Bay of Fundy, based on relationships established between secondary ports and Saint John, as determined by the Canadian Hydrographic Service, was carried out which suggested a maximum astronomical tide at Truro of 9.45 m. These data are reproduced in Figure 4.4.

Dillon-Horner pointed out that the extrapolation of tides which exceed the elevation of tidal dykes along the Salmon River Estuary (9.9 m) is highly questionable, and for this reason primarily, suggested that the downstream boundary of any hydrodynamic model be situated well out into Cobequid Bay at Bass River (which would be similar to Burntcoat Head in terms of tidal regime).

ii) Meteorological Effects on Tides

Based on analysis of atmospheric pressure data at Truro, Dillon-Horner concluded that the maximum drop in pressure was 5 kilopascals which according to their calculations, corresponded to an increase of 0.5 m in stage on the Bay of Fundy. It was pointed out that, if a pressure drop moved across the Bay of Fundy at the same speed as the rate of travel of a free oscillation (96 km/hr), the effect of a pressure drop on tidal elevations would be amplified, although no supporting calculation was undertaken.

A frequency analysis of hourly wind speed data as recorded at Truro, Debert, Greenwood and Saint John was carried out, the results of which are summarized in Table 4.1.

Based on an analysis of Table 4.1, and considering the fetch length, wind set-up was estimated at Truro as shown on Figure 4.5. Assuming that the frequency of these events are dependent, it was concluded that the combined rise in sea level (or storm surge) can reach 1.8 m on the average once every 100 years (based on an interpretation of Figure 4.5).

4.15 Godin et al - 1981

In 1981, staff of the Marine Environmental Data Services Branch, Marine Sciences and Information Directorate, Fisheries and Oceans Canada, published a report entitled, "Statistical Search for High Water at Board Landing Bridge, Truro, Nova Scotia", based again on the data recorded at the site in 1971 and 1972 (Godin et al, 1981). Regression equations relating stage at Truro were developed based on both predicted and observed stage at Saint John.

The appropriate relationship, based on predicting stage at Saint John, is shown graphically on Figure 4.6.

A statistical analysis of the astronomical tide at Saint John was carried out by Godin et al (1981) resulting in the following:

1. spring Tide - 7.96 m - twice a month
2. perigean Tide - 8.11 m - once a month
3. spring Perigean Tide - 8.63 m - once every 13 months
4. declinational Spring Perigean Tide - 8.87 m - once every 18.6 years.

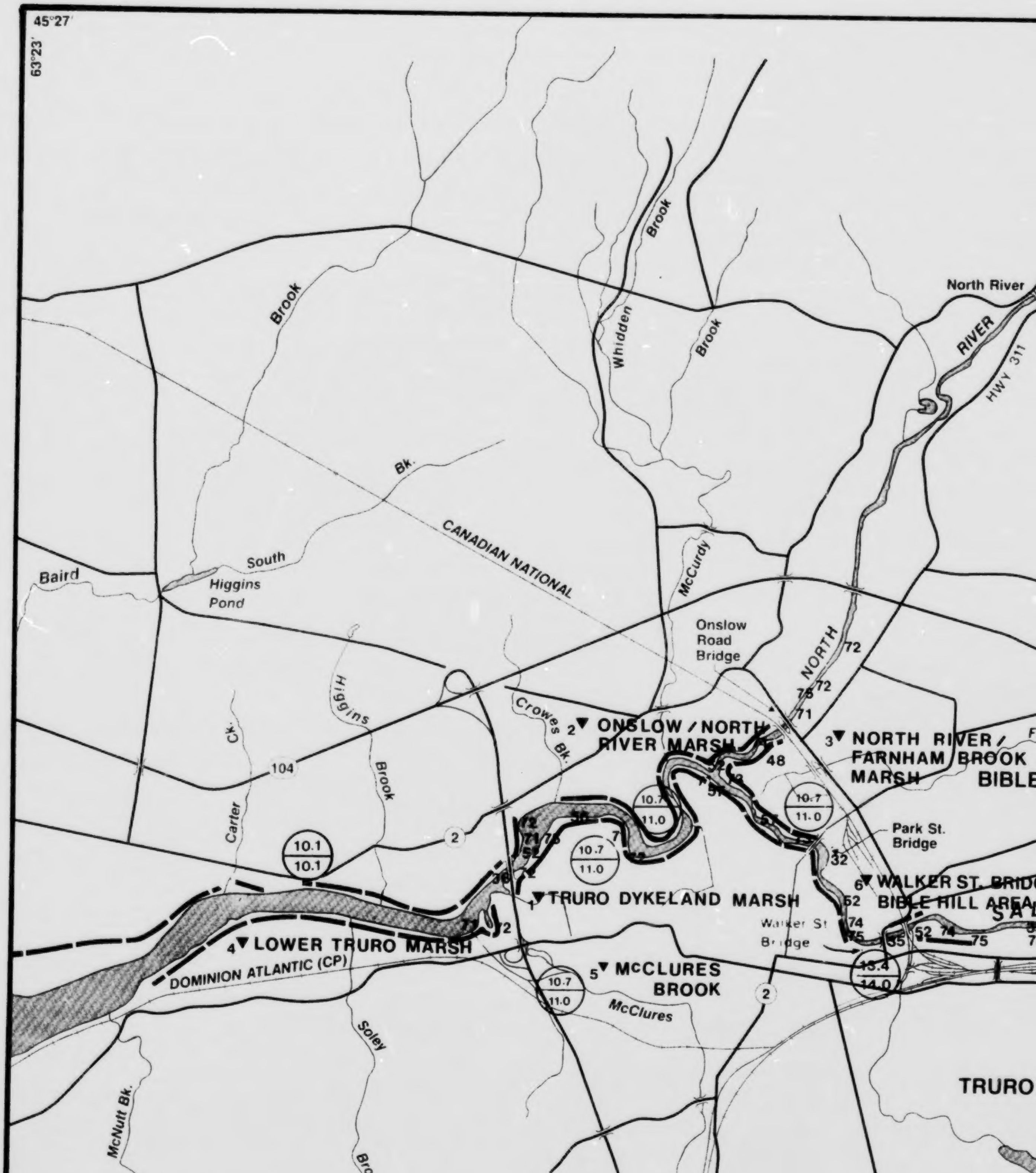
TABLE 4.1
Annual Maximum Wind Speed
(Kilometres per Hour)

Station	Maximum Observed Hourly	Estimated Frequency in Years 10	30	50	100	Probable Maximum Hourly	Probable Maximum Gust
Truro	93	92	108	118	127	130	-
Debert	77	101	119	124	134	109	-
Greenwood	113	103	119	126	135	154	161
Saint John	97	90	101	105	111	134	145

Source (Dillon-Horner, 1980)

(The period of record analysed was unspecified in the report)

The analysis by Godin et al went further than those which preceded it in that an analysis of residuals was carried out; the residual being the difference between observed stage at Board Landing Bridge and that which would be predicted by the equation. It was assumed that the residual was the product of a meteorological disturbance(s). It is noteworthy that residuals in excess of one metre were observed during the period of record at the Board Landing Bridge which was less than 14 months. The relevant results of the analysis are shown on Figure 4.7. Because of the relatively short period of record and the presence of outliers, the investigators chose to select a return period of five months in their analysis of joint probability of astronomical and meteorological tides. The reason given for this was that at a return period of 1:5 months, the outliers in the analysis would have little impact on the estimate of the residual. Based on their analysis, the 1:90 year high tide was estimated between 10.1 and 10.25 m, while a 1:180 year high tide was estimated between 10.2 and 10.45 m. Based on a slightly different combination, it was concluded that a level of 10.2 m would have a return period of 1:16 years. (This points out the obvious difficulties with such a small set of recorded data.)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McCurdy
Onslow
Road
Bridge
NORTH
72
78 72
71
Higgins
Crowes Bk.
2 ▼ ONSLOW / NORTH RIVER MARSH
3 ▼ NORTH RIVER / FARNHAM BROOK MARSH
BIBLE
48
Ck.
104
Brook
Carter
10.7
11.0
10.1
10.1
2
Park St.
Bridge
32
6 ▼ WALKER ST. BRIDGE
BIBLE HILL AREA
1 ▼ TRURO DYKELAND MARSH
52
Walker St
Bridge
74
75
4 ▼ LOWER TRURO MARSH
DOMINION ATLANTIC (CP)
5 ▼ McCLURES BROOK
McClures
13.4
14.0
Soley
TRURO
McNutt Bk.
Brook
Brook
102
SCALE
Metres 1 000
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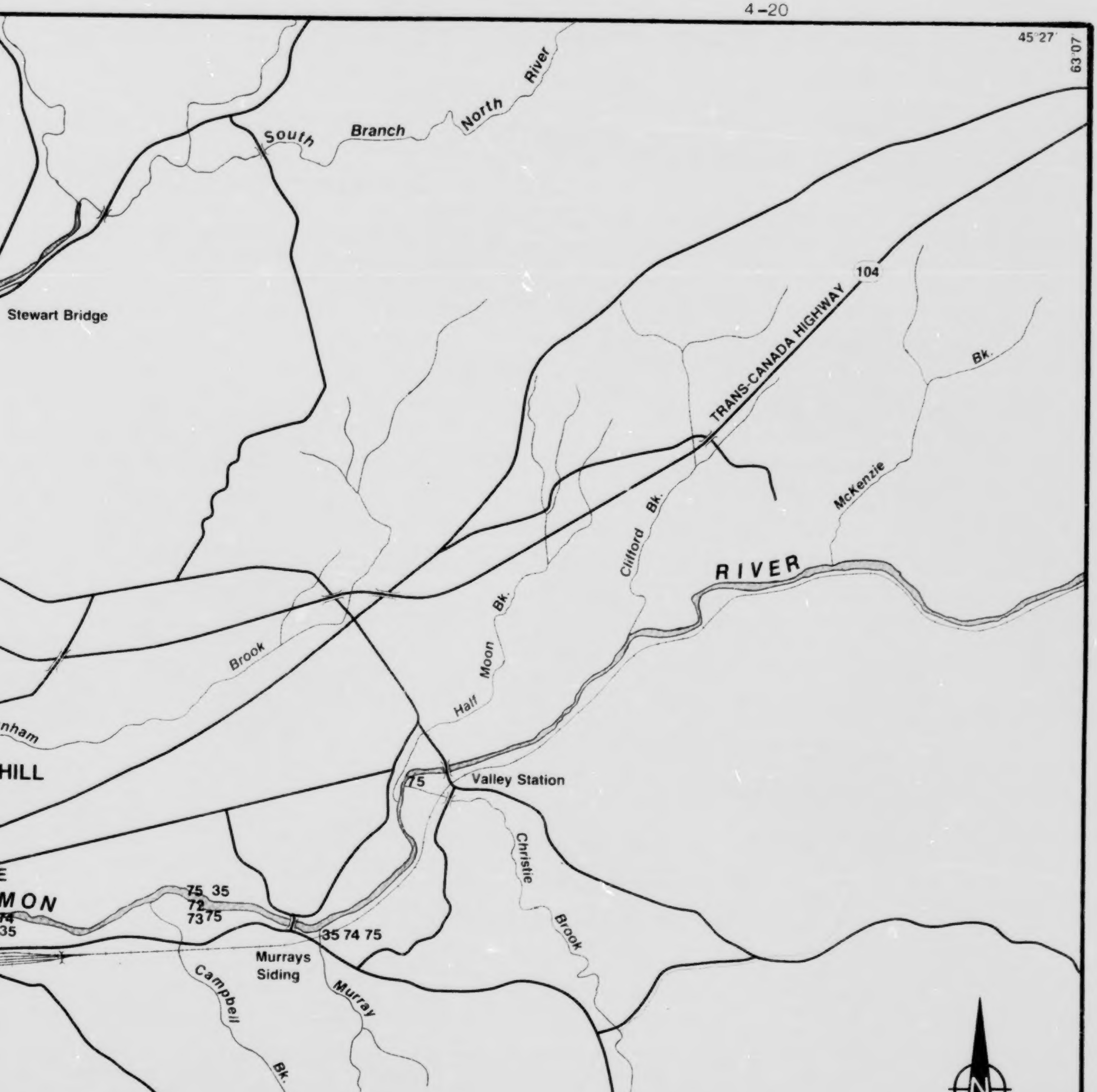
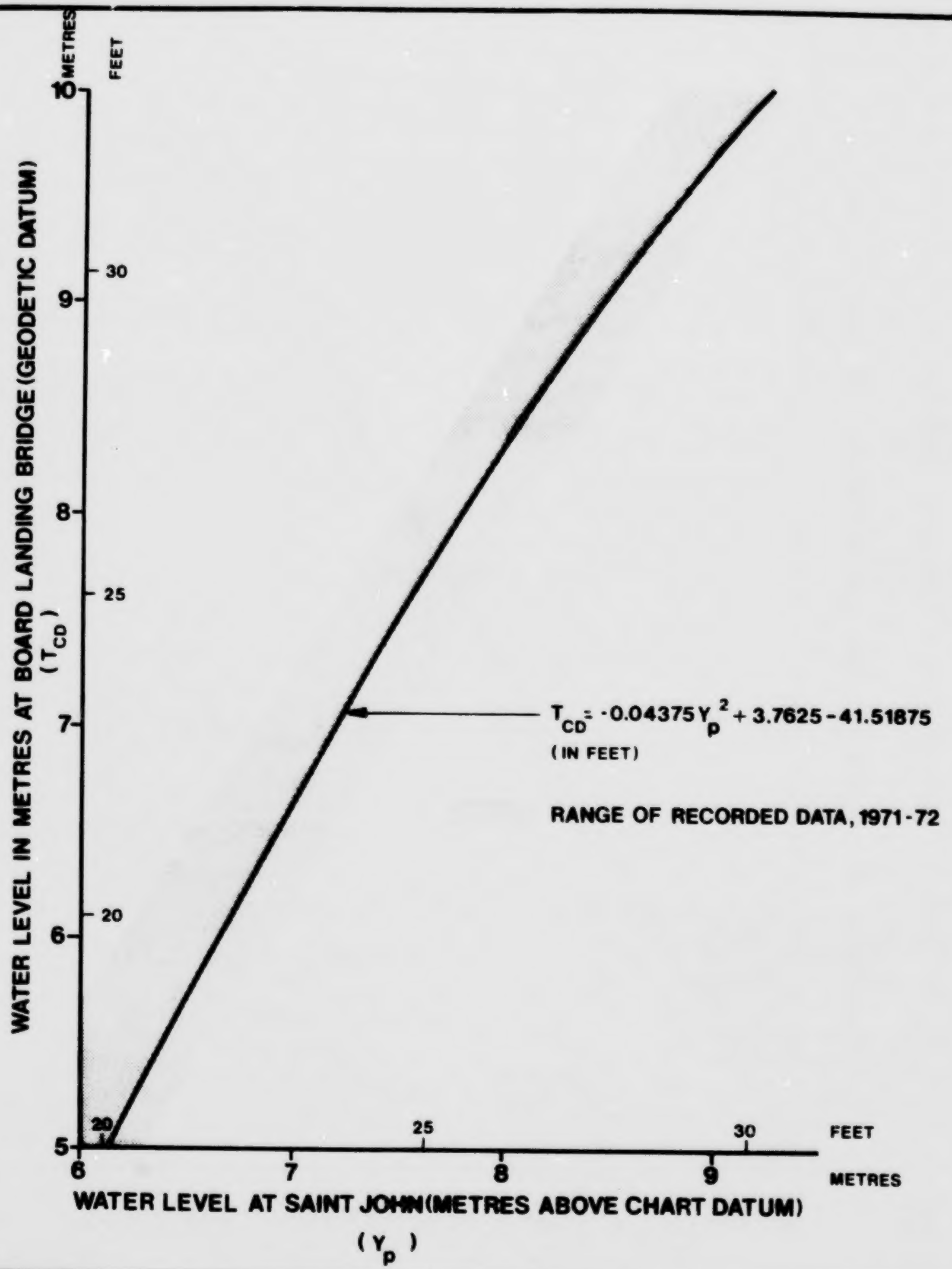
1 000
2 000
3 000 Metres
63°23′
45°20′


45°27'
63°07'
South Branch North River
Stewart Bridge
104
TRANS-CANADA HIGHWAY
Bk.
McKenzie
Clifford Bk.
RIVER
Half Moon Bk.
Brook
nham
HILL
75
Valley Station
Christie Brook
E
MON
75 35
72
73 75
74
35
35 74 75
Murrays Siding
Campbell Bk.
Murray
Lepper
Brook
N
LEGEND
75 ICE JAM OCCURED IN 1975 AT THIS LOCATION ETC
1▼ ONE OF SIX REPRESENTATIVE AREAS FOR DETERMINATION OF FLOOD LEVELS
1:20 10.7 11.0 1:100 ESTIMATED 1:20 AND 1:100 YEAR FLOOD LEVEL
SOURCE: CARSON, 1978

SUMMARY OF ICE JAM ANALYSES AT TRURO

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 4.1

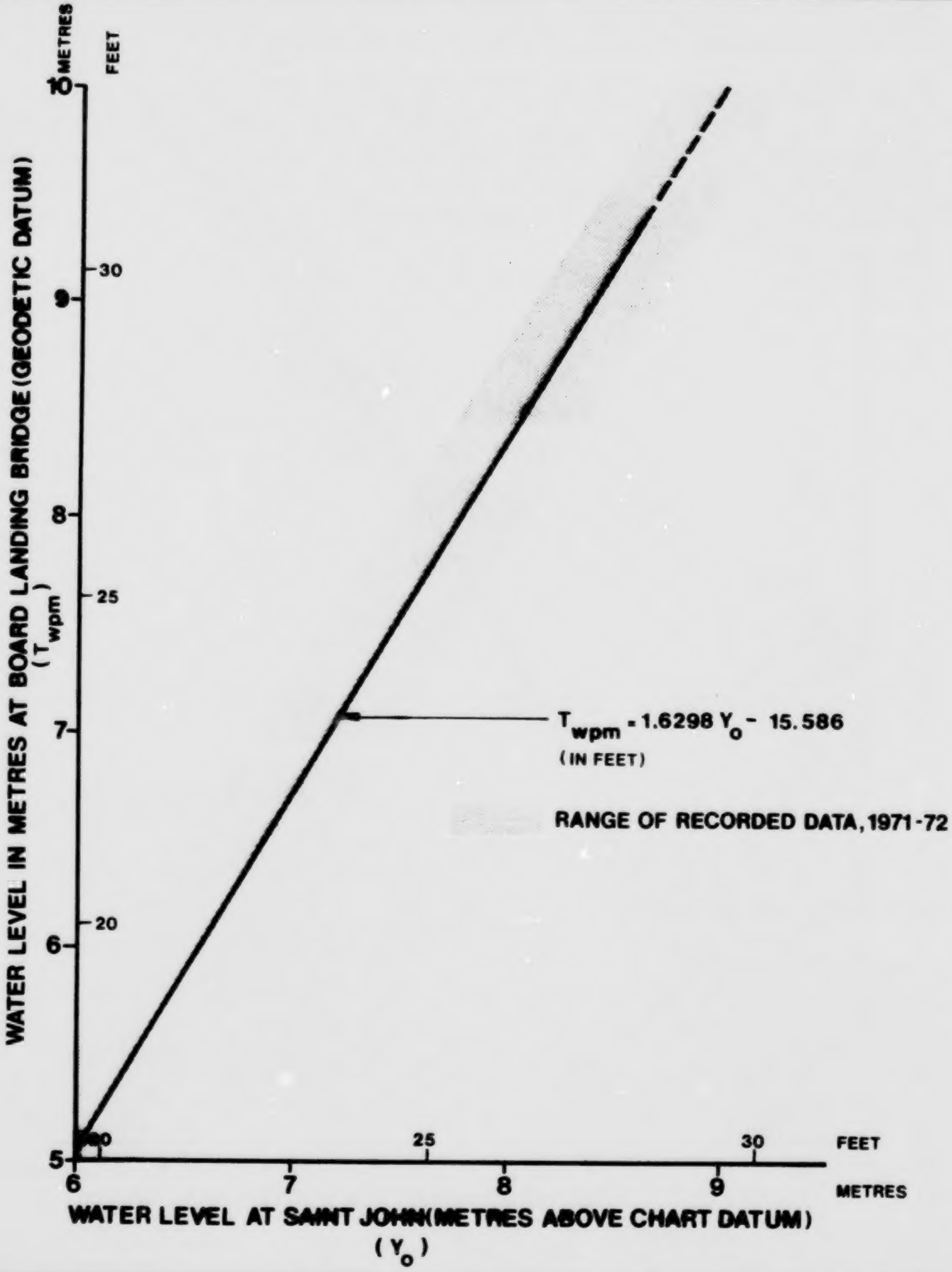


SOURCE: DESPLANQUE, 1977

TIDAL ELEVATION AT TRURO VS. SAINT JOHN

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 4.2

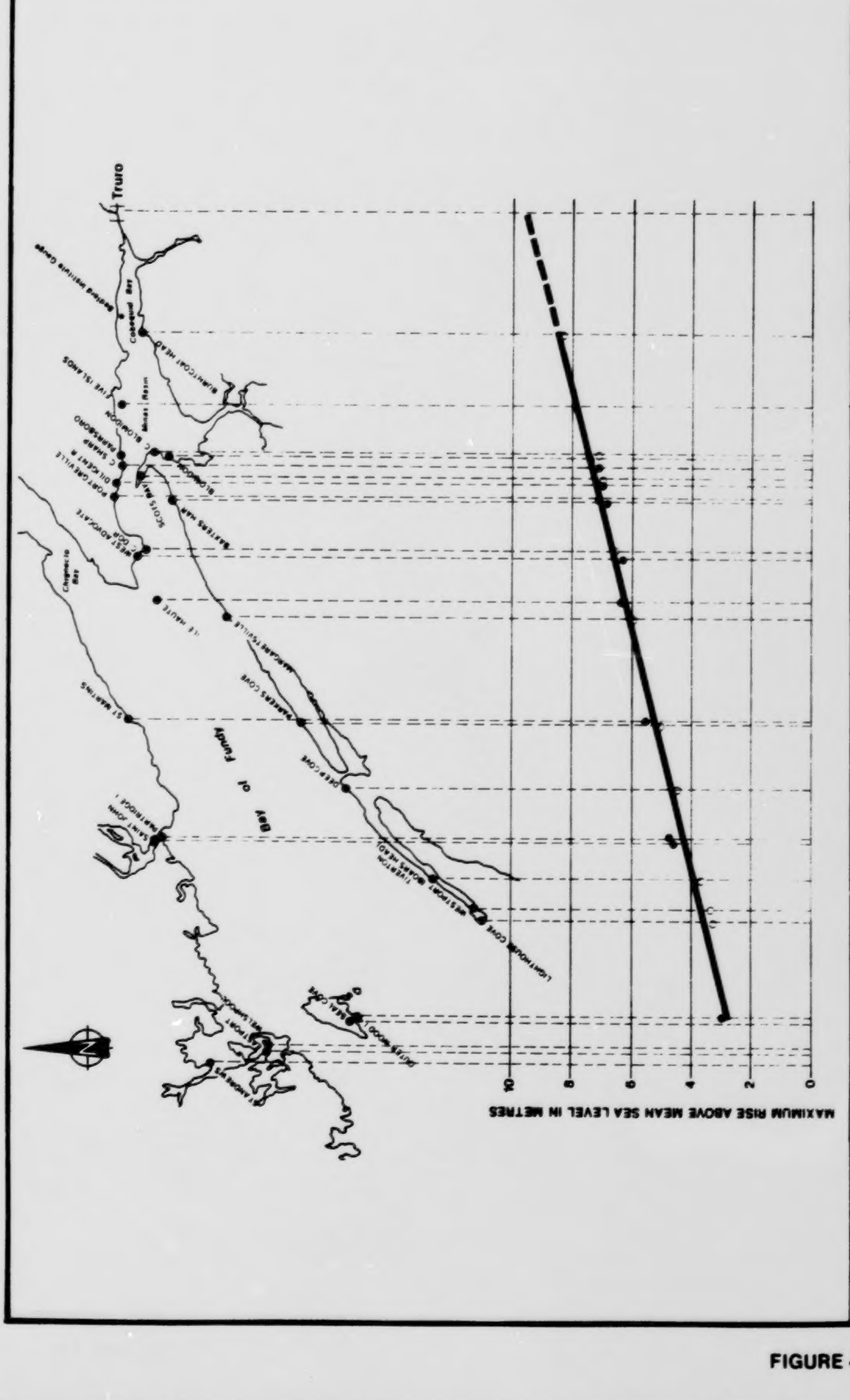


SOURCE: INLAND WATERS DIRECTORATE, 1978

TIDAL ELEVATION AT TRURO VS. SAINT JOHN

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 4.3



MAXIMUM RISE ABOVE MEAN SEA LEVEL- BAY OF FUNDY

LEGEND

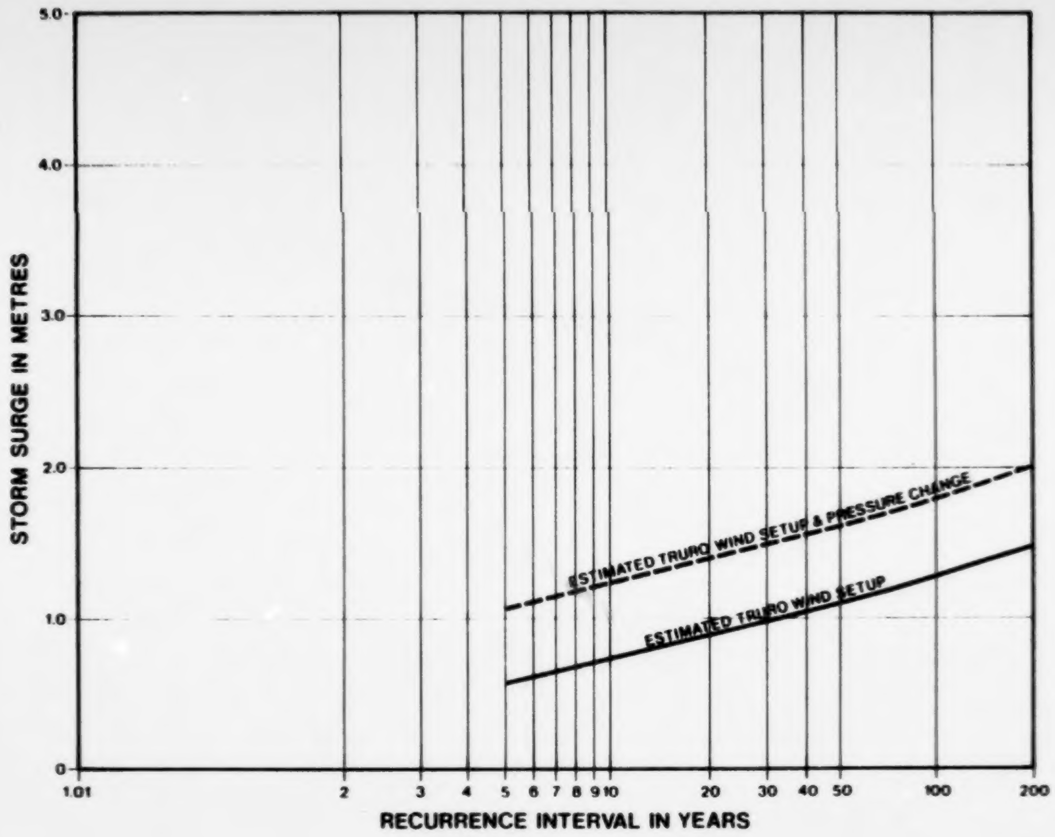
- ● NORTH SHORE
- ○ SOUTH SHORE

SOURCE : CANADIAN HYDROGRAPHIC SERVICE ANNUAL PUBLICATION / DILLON HORNER 1980

SCALE
km 0 10 20 30 km

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 4.4

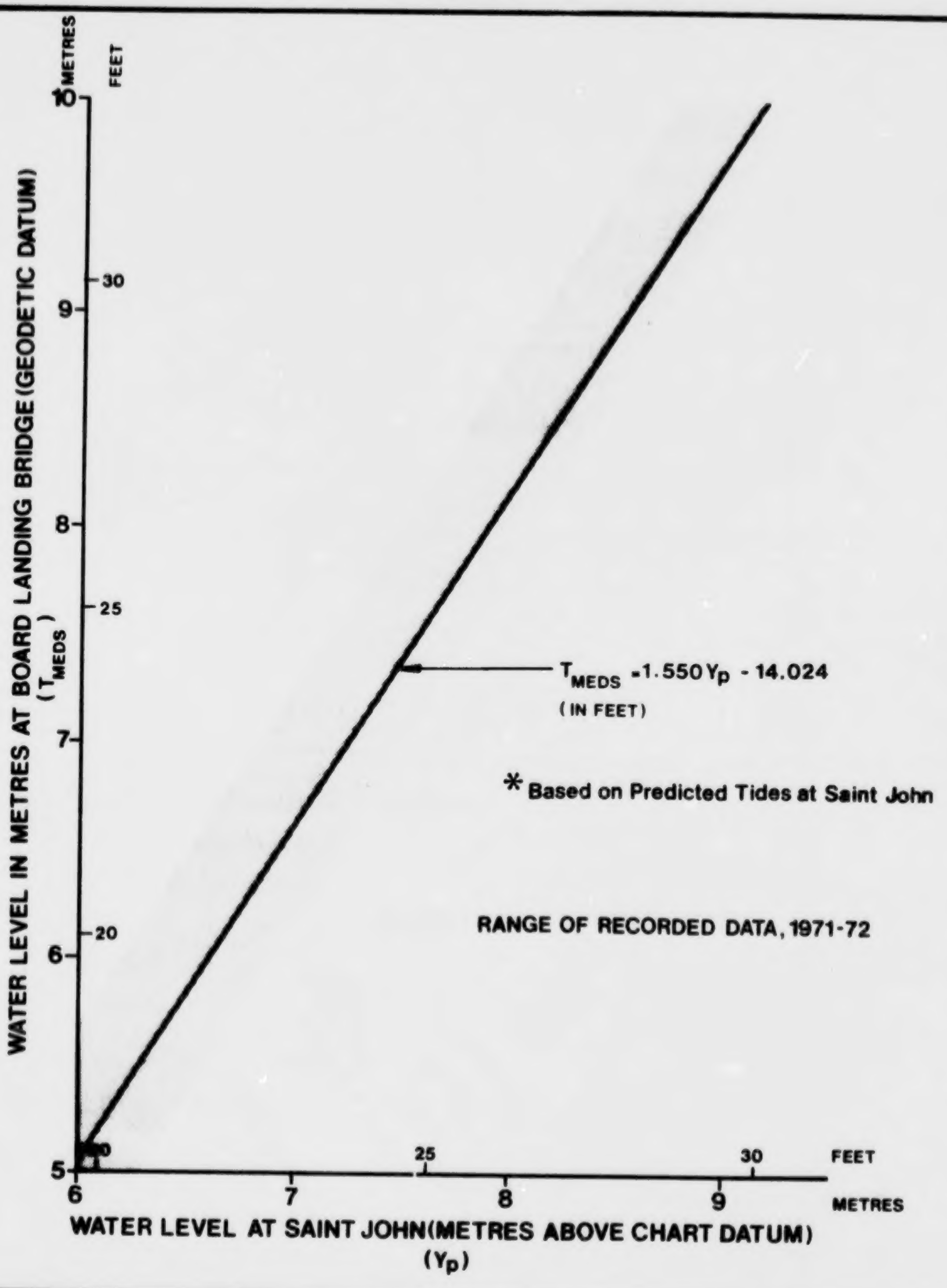


CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

SOURCE : DILLON-HORNER (1980)

ESTIMATED
STORM SURGE STATISTICS
AT TRURO

FIGURE 4.5



$T_{MEDS} = 1.550\,Y_p - 14.024$
(IN FEET)

* Based on Predicted Tides at Saint John

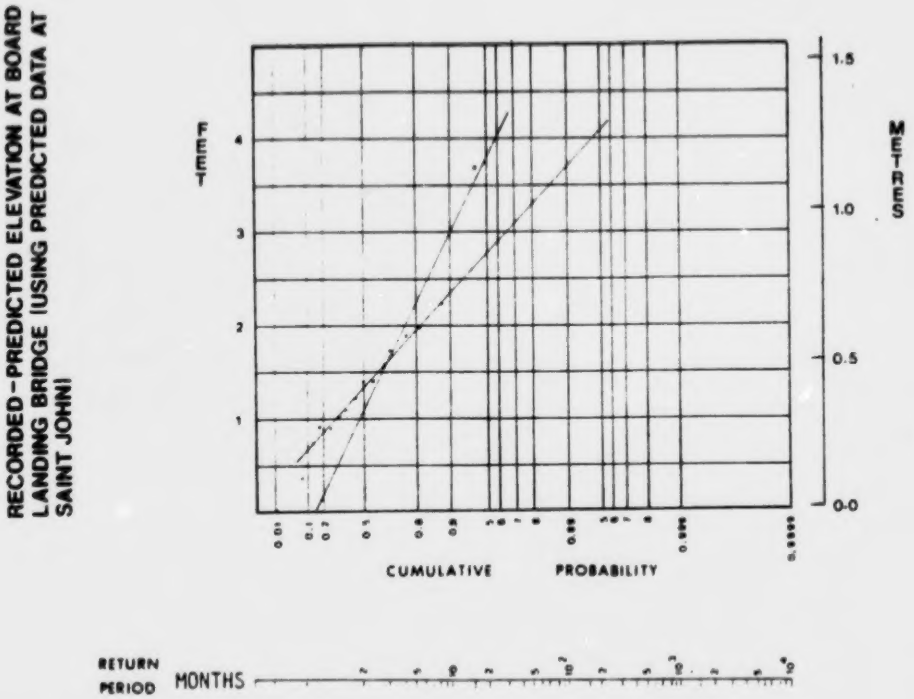
RANGE OF RECORDED DATA, 1971-72

SOURCE: GODIN ET AL, 1981

TIDAL ELEVATION AT TRURO VS. SAINT JOHN

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 4.6



SOURCE: GODIN ET AL, 1981

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

RECORDED-PREDICTED ELEVATION AT BOARD LANDING BRIDGE

FIGURE 4.7

5.0 FIELD SURVEYS

5.1 General

The successful application of the selected hydraulic model required accurate physical and hydrometric information describing the hydraulic characteristics of the study area.

The required physical information included the following:

1. width, depth, shape and elevation (geodetic datum) of the rivers at specified cross-sections (including the adjacent floodplain)
2. a description of the riverbed and adjacent floodplain flow paths in relation to surface roughness, rock outcrops, gravel bars, density and nature of vegetation.
3. measurements and characteristics of hydraulic structures such as bridges, culverts, dykes, aboiteaux, etc.

The hydrometric data included:

1. water surface elevations at times of high flows and high tides
2. discharge measurements during periods of high flow.

Topographic information on flood plain channels was obtained, for the most part, from topographic maps rather than field surveys. Flow rates were obtained from Environment Canada records maintained by the Inland Waters/Lands Directorate.

Section 5.2 discusses the hydrometric data collection program and Section 5.3 outlines the methodology and results of the physical data collection program. A summary discussion of results and level of accuracy, etc. is given in Section 5.4.

The use of these data in the set-up, calibration and verification of the hydrodynamic model is discussed in Section 8.0.

5.2 Hydrometric Data Collection Program

5.2.1 Gauge Readings

The hydrometric data collection program was initiated by Dillon-Horner in the spring of 1980 and subsequently reactivated by staff of Environment Canada and the Province of Nova Scotia during peak flow periods in 1982.

Gauge locations were selected with a view to getting water level readings from representative locations distributed along the length of the rivers. Accessibility at times of high water was a factor and this resulted in use of bridges and structures near roads when otherwise appropriate. The risk of damage from vandals or ice was a consideration as well. An overriding consideration was to mount gauges in order to obtain representative readings that were not affected by local conditions such as standing waves or local backwater.

Staff gauges were installed at the locations shown on Figure 5.1 and listed in Table 5.1. The time and magnitude of the water levels were recorded manually by field observers. All readings were converted to GSC datum during the field survey program discussed in Section 5.3. Water levels measured during the field program are summarized in Table 5.1 and additional data are given in Appendix A. The relevant resulting measured stage hydrographs are compared to simulation results from the hydrodynamic model in Section 8.0.

5.2.2 Flow Resistance

In order to assess the roughness coefficient (Manning's "n") along the channels of the rivers and floodplains, a field inspection was

TABLE 5.1

List of Staff Gauge Locations and Water Levels (m)

Gauge Location No.	Name	April 16/80 (b) Elev.	Hour	April 29/82 (a)	Remarks (c)
S1	Cutten Bk.	9.55	01:33	-	Lower Truro on Cutten Brook
S2	McClures Br.	-	-	6.71	Lower Truro, on timber of McClures Brook aboiteaux by survey)
S3	Board Ldg. Br.	9.65	01:35	6.95	Fastened to old bridge abutment crib near Palliser Restaurant
S4	McCurdy Bk.	9.69	01:51	6.89	Onslow, McCurdy Brook aboiteaux on river side of dyke; C4 on power pole near S4
S6	Farnham Bk.	-	-	7.46	Bible Hill area, on river side of Farnham Brook aboiteaux
S7	Park Street	9.92	01:55	8.90	Bible Hill area, on crib for north pier of old Park Street bridge
S8	CNR Bridge	-	-	11.44	On south abutment, upstream side of bridge
S9	Gillis Farm	-	-	16.44	On Gillis property
S10	Murray Siding Bridge	-	-	21.31	On north (downstream side) of crib abutment
S11	Valley Br.	-	-	28.67	Reference mark on Valley Bridge (Chain Drop)
N1	Railway	-	-	7.41	North River on small CNR aboiteaux, west bank downstream of CNR bridge
N2	Bailey Br.	-	-	3.37	North River near Upper Onslow and Marshall's Corner
SB	Bass River	8.7	01:20	-	
	Stewart Br.	-	-	25.89	I.W.D. gauge 01DH004

a) at 18:00 hours; measurements by Environment Canada and Province of Nova Scotia

b) measurements by Dillon-Horner

c) Refer to locations on Figure 5.1

made by engineers and surveyors responsible for developing the computer model. The resulting roughness coefficients are summarized and discussed in more detail in Section 8.0.

5.3 Physical Surveys

5.3.1 Methodology and Equipment

Subsequent to the initial installation of staff gauges by the consultant in 1980, the gauge elevations were referenced to Geodetic Survey of Canada (G.S.C.), using the services of a Nova Scotia Land Surveyor. Circuits were run with levelling rod and spirit level to reference points at each site. Refurbishing and resurveying the gauge network, necessitated by the ravages of winter ice and vandals, was done by Nova Scotia Department of the Environment staff in 1982.

Cross-sections were surveyed along the rivers at about 1000 m spacing in the summer of 1982 by staff of the Nova Scotia Department of the Environment, using a theodolite for determining elevation and distance. Details of the survey procedure are given in Appendix A. All cross-sections were located in the field, using aerial photography at a scale of 1:5472, by staff of the hydraulic modelling team in order to provide the best possible representation of the river and floodplain channels. (The aerial photography is available as supplementary information.) The cross-section locations are given on Figures 5.1a and 5.1b.

Level circuits to tie each cross-section to geodetic elevations were made by the Nova Scotia Department of the Environment using Wild NA-2 and NAK-2 automatic levels and Frisco three-section wooden levelling rod, graduated in feet and hundredths.

Subsequent to these surveys, an error became apparent in 1985 which necessitated resurveying a number of cross-sections (see Appendix A). This was subsequently carried out by a Nova Scotia Land

Surveyor and the appropriate corrections were made to the initial survey results.

Typical cross-sections are shown on Figures 8.4 to 8.7. A complete inventory of computer plotted cross-sections is available as supplementary information. The results of the surveys were also used to plot the river bottom profile (thalweg) which is given on Figures 8.2 and 8.3.

5.3.2 Hydraulic Structures

The physical characteristics of hydraulic structures were obtained by field measurements and from drawings supplied by owners including the Nova Scotia Department of Agriculture, the Nova Scotia Department of Transportation and the C.N.R. All bridges were measured for hydraulic dimensions, distance between piers, number and width of piers and bottom chord and roadway elevation. Tables 5.2 to 5.5 summarize dimensions and provide a brief description of various structures in the area that have the potential to affect flood water levels. The hydraulic structures and aboiteaux are located by reference identification numbers on Figures 5.1a and 5.1b.

5.4 Discussion of Results

5.4.1 Hydrometric Data

Over the duration of the field survey program it was possible to obtain only one complete set of observations of tide and freshwater levels due to a number of factors including:

1. winter conditions with ice in the rivers (either flowing or fixed), which precluded good level data collection
2. ice in the estuary at times prevented inflow of tide water
3. lack of occurrence of heavy rainfall; for example on April 28, 1982, the forecast was that heavy rain would last all day on the 29th and plans were made accordingly to read gauges to

TABLE 5.2

Structures Crossing Salmon River

Location*	Span (m)	Elev.** Bott. Open. (m)	Elev.*** Top Open. (m)	Description
1. Old Board Landing Bridge - piers only, no superstructure	N/A	N/A	N/A	N.S. Dept. of Highways. Constructed in 1891. Only abutments and piers remain
2. Hwy 102 Bridge (Salmon River)	179.8	5.8	10.67	N.S. Dept. of Highways. Constructed in 1969 and 1986. Steel girder 2 x 2 lanes, 4 spans, 3 piers, steel faced
3. Park St. Bridge old (now replaced)	74.1	5.0	10.0	N.S. Dept. of Highways. Constructed in 1899. 1 steel through truss and 3 short timber beamspans
4. Walker St. Bridge (Main Street)	25.3	10.0	13.6	N.S. Dept. of Highways. Structure erected around 1971 to replace old span constructed in 1892 Steel tied arch
5. Submerged Watermain	N/A	6.4	N/A	Located on channel bed about 15.25 m upstream of Walker St. bridge. Constructed around 1972. Concrete protective cover forms weir
6. C.N.R. Bridge	61.0	9.9	12.2	Existing structure constructed in 1919. 2 span, 2 track, steel girder, stone centre pier
7. Murray Siding (Vernon Bridge)	42.7	19.7	26.2	N.S. Dept. of Highways. Constructed in 1891. 1 steel through truss span. Sloped roadway
8. Valley Bridge	36.6	25.3	32.6	N.S. Dept. of Highways. Constructed in 1923. 1 span steel through truss.

* Refer to locations on Figure 5.1
** Elevation of Bottom Opening
*** Elevation of Top Opening

TABLE 5.3

Structures Crossing North River

Location*	Span (m)	Elev. Bott.Open. (m)	Elev. Top Open. (m)	Description
9. C.N.R. Bridge	51.8	7.9	11.1	Existing structure constructed in 1919
10. Bailey Bridge Marshall's Corner	36.6	7.7	11.5	Old, now replaced at same location as 10a
10a Onslow Bridge (at Marshall's Corner)	38	7.3	11.5	1 span box girder; concrete deck; 14° skew
11. T.C.H. #104 Bridge	54	11.7	13.6	N.S. Dept. of Highways. Constructed in 1968. 2 span, steel girder, conc. centre pier, 1.5 m wide
12. Stewarts Bridge	42.7	25	29.3	N.S. Dept. of Highways. Constructed in 1924, 2 span.
13. E. Lynds Bridge, Hwy. 311	32.6	31.7	32.2	2 span, timber

* Refer to locations on Figure 5.1

TABLE 5.4

Aboiteaux Structures

Brook Name		Invert Elevation (m)	Width (m)	Depth (m)	Description
A1	Cutten Brook	5.8	.76	.46	
A2	Soley Brook	5.2	.9	1.2	
A3	McClures Brook	3.35	1.83 x 3	-	3 conc. pipes, 37 m long sloping 0.3 m, external hanging flaps at outlets
A4	Hunter Drain	5.6	1.07	-	
A5	Farnham Brook	6.5	1.07 x 3	0.76	
A6	CNR Aboiteau	6.2	0.91	-	West Bank, North River downstream of CNR bridge timber culvert with flap
A7	McCurdy Brook	5.9	1.83 x 3	-	3 concrete conduits, external hanging flaps 18.9 m long conduits
A8	Higgins Brook	4.75	1.1	0.8	Timber box conduit

TABLE 5.5

Other Hydraulic Structures

14 Viaduct under Hwy. 102, south of Salmon River
    Span 71.3 m between abutments, 67.7 m clear
    Elev. bottom side of structure 12.4 m
    Elev. bottom opening (land) 8.4 m
  Structure is on rows of columns: 7 rows (8 spans)

15 Passage under CNR embankment, west of North River, near Onslow Road
  D/S end is sandstone blocks, 1.8 m high, .9 m wide, and U/S end is a 2.13 x 1.62 Arch C.M. Pipe
  Invert elev. is 8 m (upstream) and 7.7 m (downstream)
  An additional 0.9 m C.M. Pipe is located nearby

McClures Brook Structures

16 Highway 102 Bridge
    Span 35 m between abutments
    Elev. bottom side of structure 4.8 m
    Elev. bottom opening (land) 14.8 m

17 Princeport Road Overpass for Hwy 102
    Span 34.0 m between abutments
    Elev. bottom side of structure 9.1 m
    Elev. bottom opening (land) 15.8 m

record peak levels. However, the predicted rainfall did not occur and as a consequence, only the receding part of the hydrograph was observed

4. difficulty and hazard of manually reading gauges at night
5. personnel and time requirements to mount a full gauge reading program. (For future investigations it would likely be worthwhile to install recording water level gauges to increase the likelihood of obtaining readings of rising and falling water elevations. The capital cost is high but perhaps warranted by better quality data and the lower overall cost.)

Only one event (April 29, 1982) measured in the field provided data suitable for calibrating the hydrodynamic model.

Other events which occurred in November and December of 1980 were measured in the tidal part of the study area by Dillon-Horner. The data from these observations are included in Appendix A and in an unpublished report (Dillon-Horner, 1980).

5.4.2 Identification of Significant Factors

Significant factors affecting flood conditions were identified as a result of the field surveys.

It was noted that the channel invert in the estuary is subject to quite wide fluctuations in elevation, location and shape due to thick deposits of silt in the estuary. High freshwater flows during periods of low tides tend to flush the silt seaward leaving a lower, gravelly riverbed in the upper estuary, around Park Street and Marshall's Corner. This action results in significant variations in channel bottom elevations of as much as 2 metres.

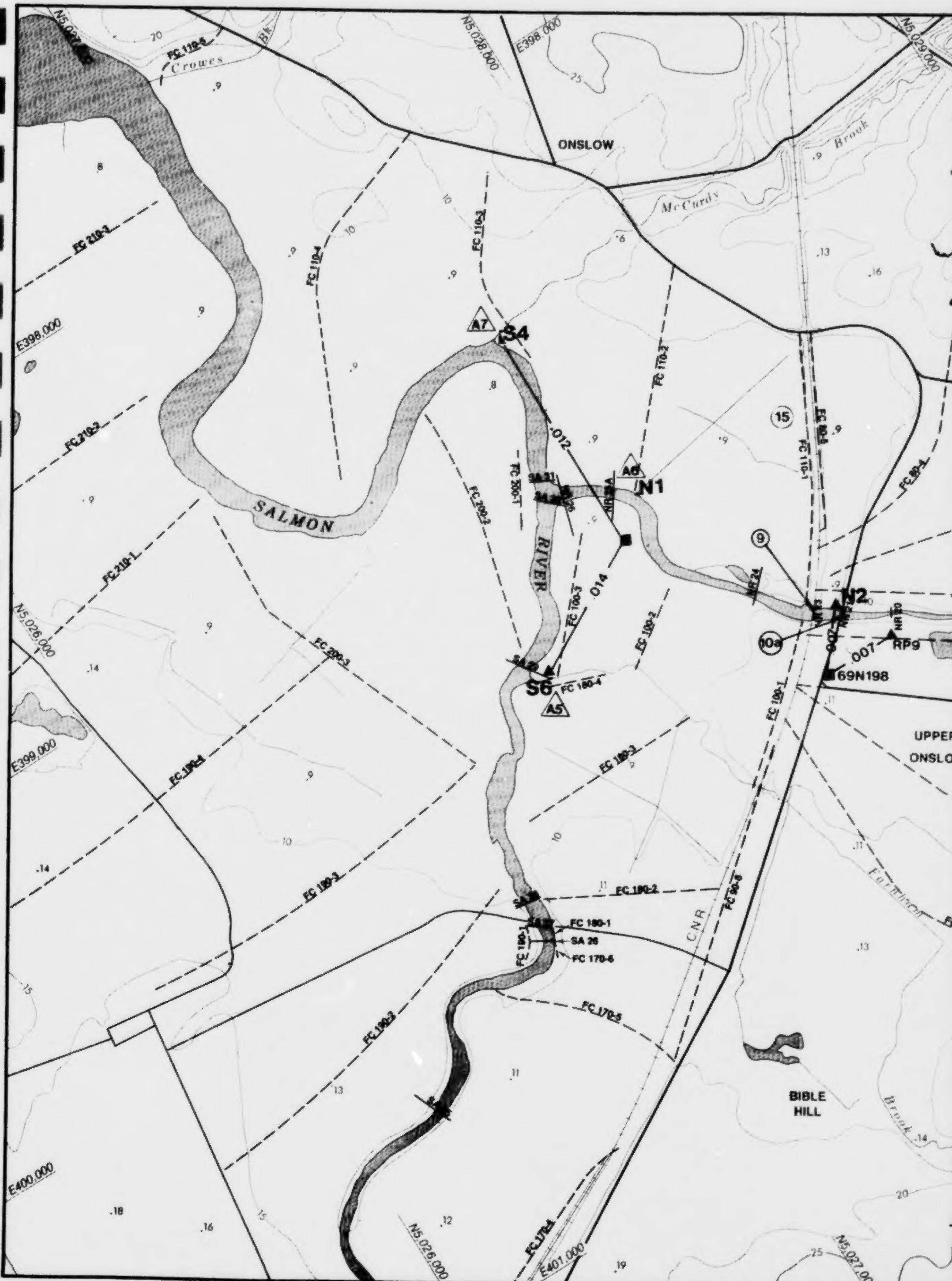
The dykes around the estuary of the Salmon River are generally built with a top elevation of 9.9 geodetic. However, the upper reaches of dykes are somewhat higher to account for backwater effects, and to contain freshet flows not related to tides. Above the confluence of the Salmon and North Rivers, the north side dyke on the Salmon River is 0.3 m lower than that on the south side. This causes overflow to the north before flows overtop on the south. The dyke on the south side upstream of Park Street is higher again and rises gradually from there to Walker (Main) Street to about 12.2 m. Since 1982 dykes have been extended on the north side of the river above Walker Street. They extend to the high bank area above the CNR bridge. On the south (Truro) side, a dyke has been constructed extending some 1500 m upstream of the CNR near Queen Street which affects flow patterns and ice but since there is no control structure at Lepper Brook, does not affect flood elevations behind the dykes.

Aboiteaux are a feature of the tidal estuary and dyked land. These structures are comprised of sluices with gates that permit flow in a seaward direction only. Thus when tidewater rises, the flap gates close and restrict inflow. When the tide falls, the head of fresh water backed up by the closed gate forces the gate open and flow towards the sea is resumed. The aboiteau generally operate in an effective manner, although some tide waters enter the streams carrying silt in suspension from time to time.

5.4.3 Accuracy of Surveys

The accuracy of geodetic surveys was independently checked due to problems identified in the area downstream of Murray Siding Bridge (SA-17). This work resulted in some revisions to cross-sections in this area. The levelling accuracy of the check surveys was all found to be within stringent limits (see Appendix A). The levelling accuracy of the original work was also found to be generally within these limits and was definitely within the level of accuracy needed for the hydraulic modelling.

The accuracy of survey loops is indicated on Figure 5.1 and discussed in more detail in Appendix A.


N5,028,000
E398,000
N5,029,000
FC 110-5
Crowes Bk
ONSLOW
McCurdy Brook
FC 210-3
FC 110-4
FC 110-2
FC 110-3
E398,000
A7
S4
15
FC 110-1
FC 80-4
.012
A6
N1
FC 210-2
SALMON
RIVER
FC 200-2
FC 200-1
SA 31
SA 30
FC 210-1
.014
FC 100-3
FC 100-2
NR 24
9
N2
NR 20
RP9
10a
.007
69N198
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FC 200-3
SA 29
S6
FC 180-4
A5
UPPER ONSLOW
E399,000
FC 190-4
FC 180-3
FC 100-1
CNR
FC 190-3
FC 180-2
SA 28
SA 27
FC 180-1
SA 26
FC 170-6
FC 190-1
FC 170-5
FC 190-2
BIBLE HILL
Brook
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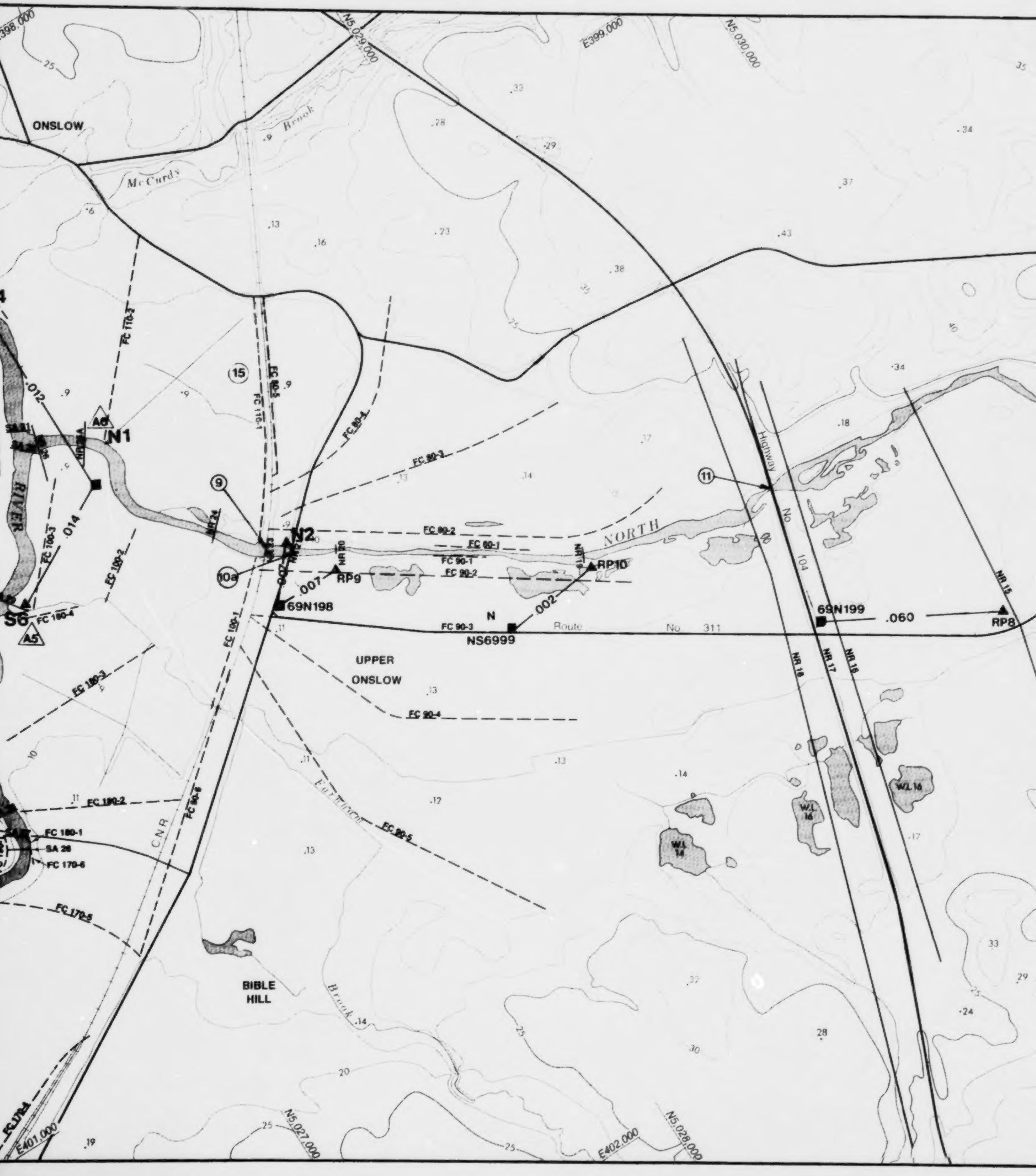
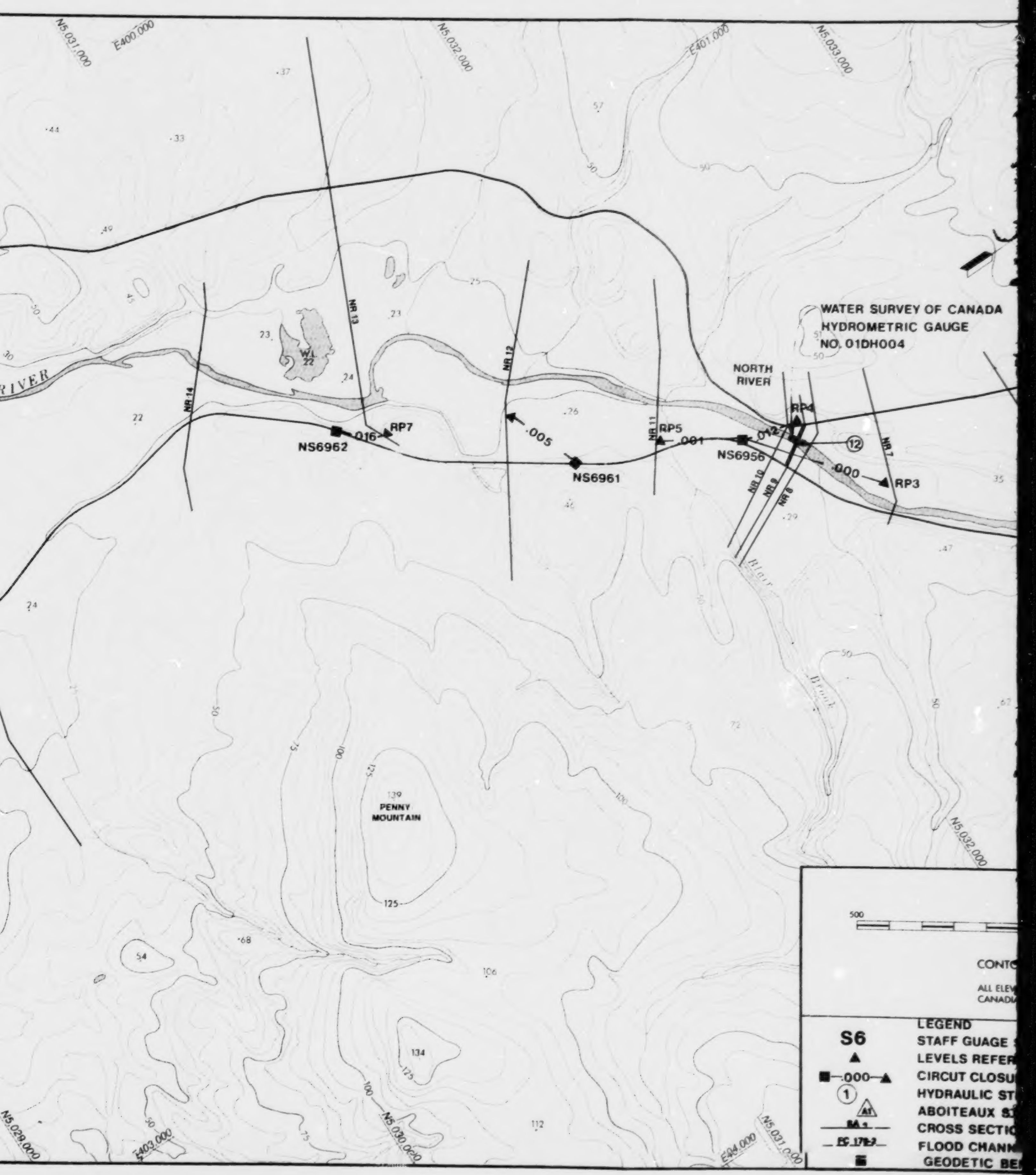
N5,026,000
FC 170-4
E401,000
N5,027,000



398,000
ONSLOW
McCurdy
Brook
N5,029,000
E399,000
N5,030,000
NORTH
RIVER
N1
A6
N2
S6
A5
RP9
RP10
RP8
69N198
69N199
NS6999
.012
.014
.007
.002
.060
⑨
⑩a
⑪
⑮
FC 170-3
FC 90-5
FC 110-1
FC 90-4
FC 90-3
FC 90-2
FC 90-1
FC 190-2
FC 190-3
FC 180-3
FC 180-4
FC 190-1
FC 180-2
FC 180-1
FC 170-6
FC 170-5
FC 170-4
FC 90-6
FC 90-5
NR 24
NR 19
NR 18
NR 17
NR 16
NR 15
SA 26
Highway
No. 104
Route No 311
UPPER
ONSLOW
BIBLE
HILL
CNR
Brook
W.L. 14
W.L. 15
W.L. 16
E401,000
E402,000
N5,027,000
N5,028,000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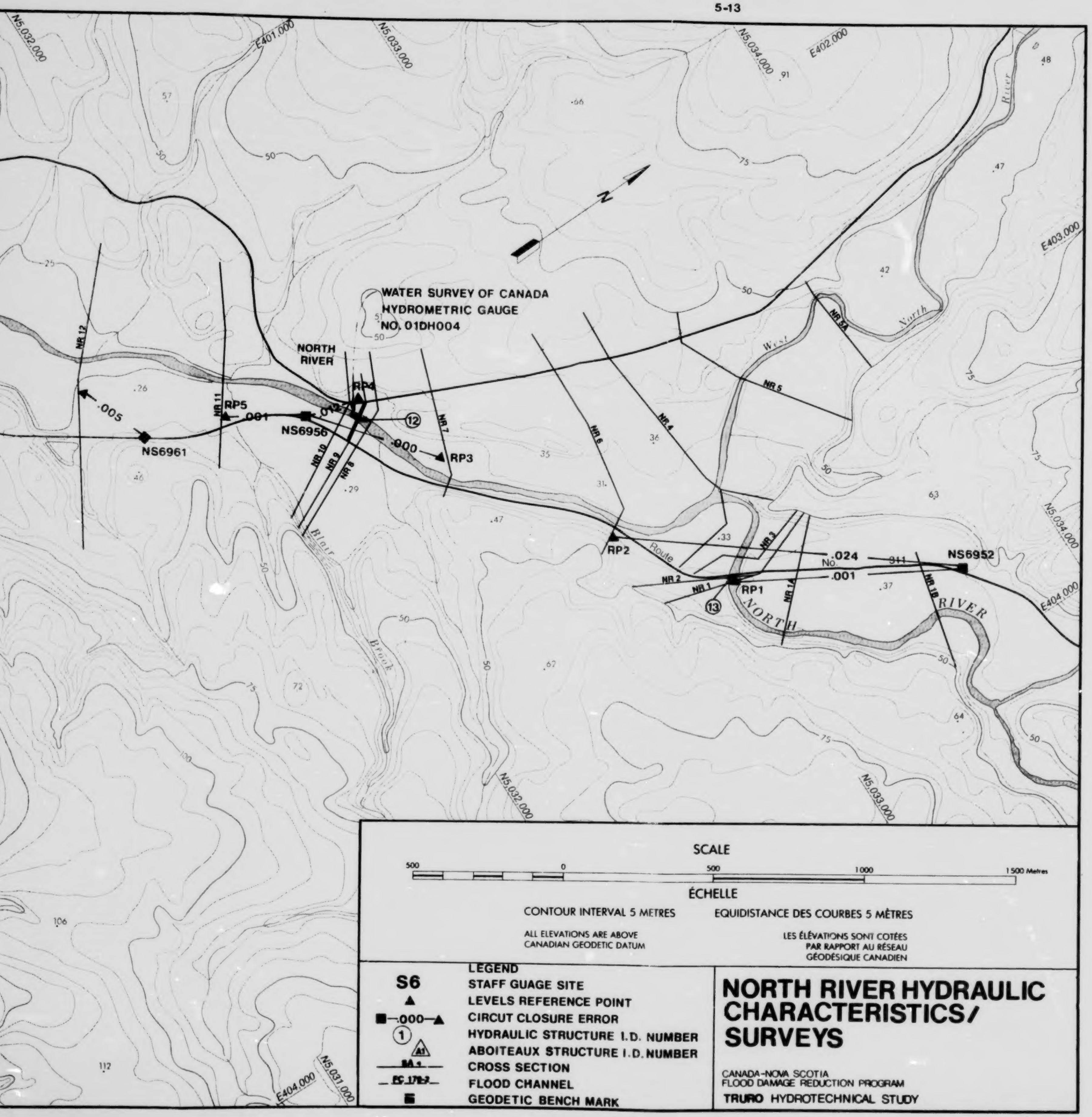
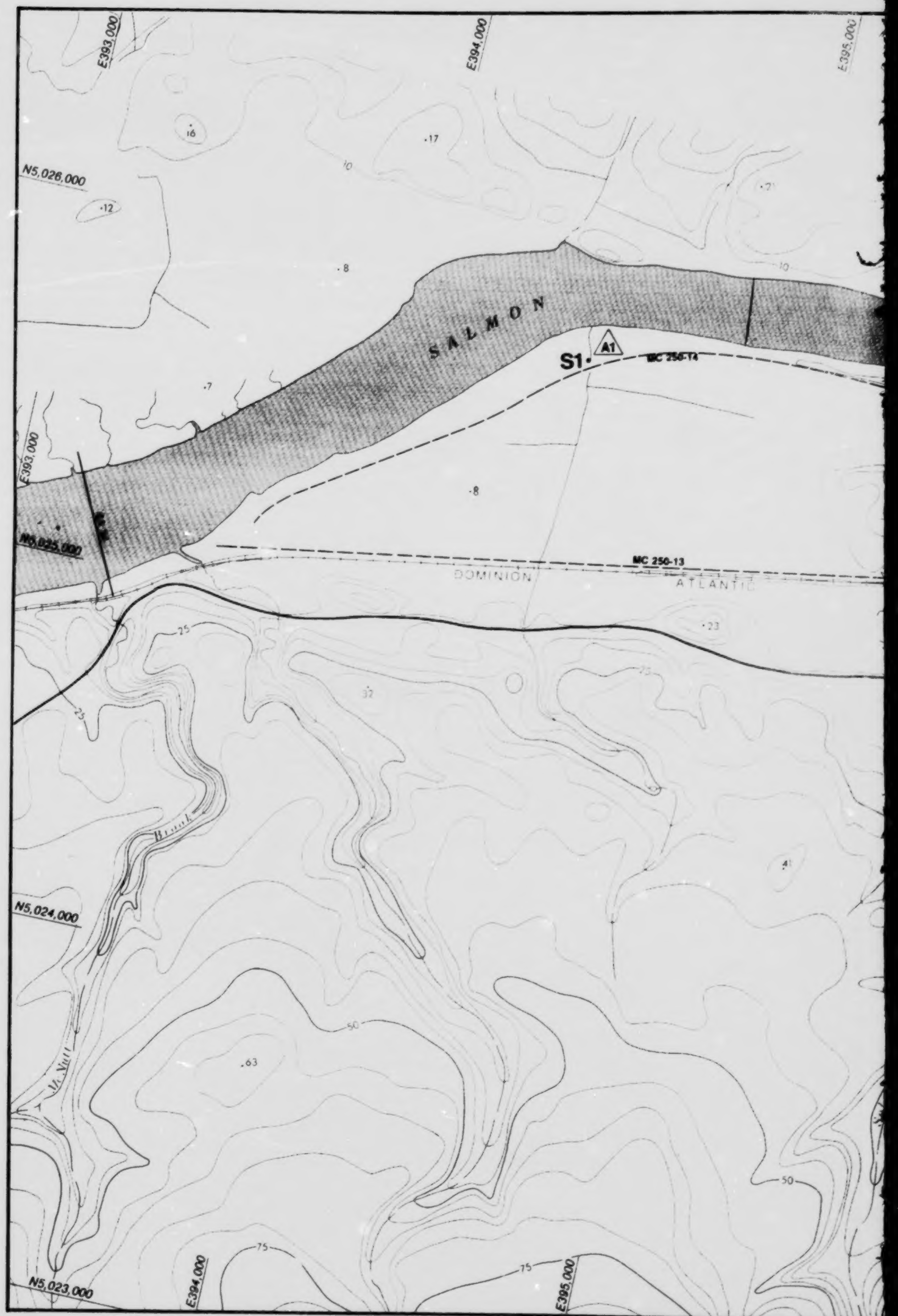
RIVER
NORTH RIVER
WATER SURVEY OF CANADA
HYDROMETRIC GAUGE
NO. 01DH004
W.L. 22
NR 14
NR 13
NR 12
NR 11
NR 10
NR 9
NR 8
NR 7
NS6962
.016
RP7
.005
NS6961
RP5
.001
NS6956
.012
RP4
12
.000
RP3
Blair Brook
PENNY MOUNTAIN
139
125
100
75
50
134
106
112
68
54
N5.029.000
E403.000
N5.030.000
E404.000
N5.031.000
N5.032.000
500
CONTO
ALL ELEV
CANADI
LEGEND
S6
STAFF GUAGE
LEVELS REFER
.000
CIRCUT CLOSU
1
HYDRAULIC ST
A1
ABOITEAUX S
8A
CROSS SECTIO
FC 178-2
FLOOD CHANN
GEODETIC BE


WATER SURVEY OF CANADA
HYDROMETRIC GAUGE
NO. 01DH004
NORTH RIVER
RP1
RP2
RP3
RP4
RP5
NS6961
NS6956
NS6952
NR 1
NR 1A
NR 1B
NR 2
NR 3
NR 4
NR 5
NR 5A
NR 6
NR 7
NR 8
NR 9
NR 10
NR 11
NR 12
.005
.001
.012
.000
.024
.001
12
13
N
NORTH RIVER
West North River
Blair Brook
Route No. 311
N5,031,000
N5,032,000
N5,033,000
N5,034,000
E401,000
E402,000
E403,000
E404,000
SCALE
500 0 500 1000 1500 Metres
ÉCHELLE
CONTOUR INTERVAL 5 METRES
EQUIDISTANCE DES COURBES 5 MÈTRES
ALL ELEVATIONS ARE ABOVE CANADIAN GEODETIC DATUM
LES ÉLÉVATIONS SONT COTÉES PAR RAPPORT AU RÉSEAU GÉODÉSIQUE CANADIEN
LEGEND
S6 STAFF GUAGE SITE
▲ LEVELS REFERENCE POINT
■—.000—▲ CIRCUT CLOSURE ERROR
① HYDRAULIC STRUCTURE I.D. NUMBER
A1 ABOITEAUX STRUCTURE I.D. NUMBER
8A CROSS SECTION
FC 176-3 FLOOD CHANNEL
■ GEODETIC BENCH MARK

NORTH RIVER HYDRAULIC CHARACTERISTICS/ SURVEYS

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY


SALMON
S1.
A1
MC 250-14
MC 250-13
DOMINION ATLANTIC
Brook
McNutt Brook
E393,000
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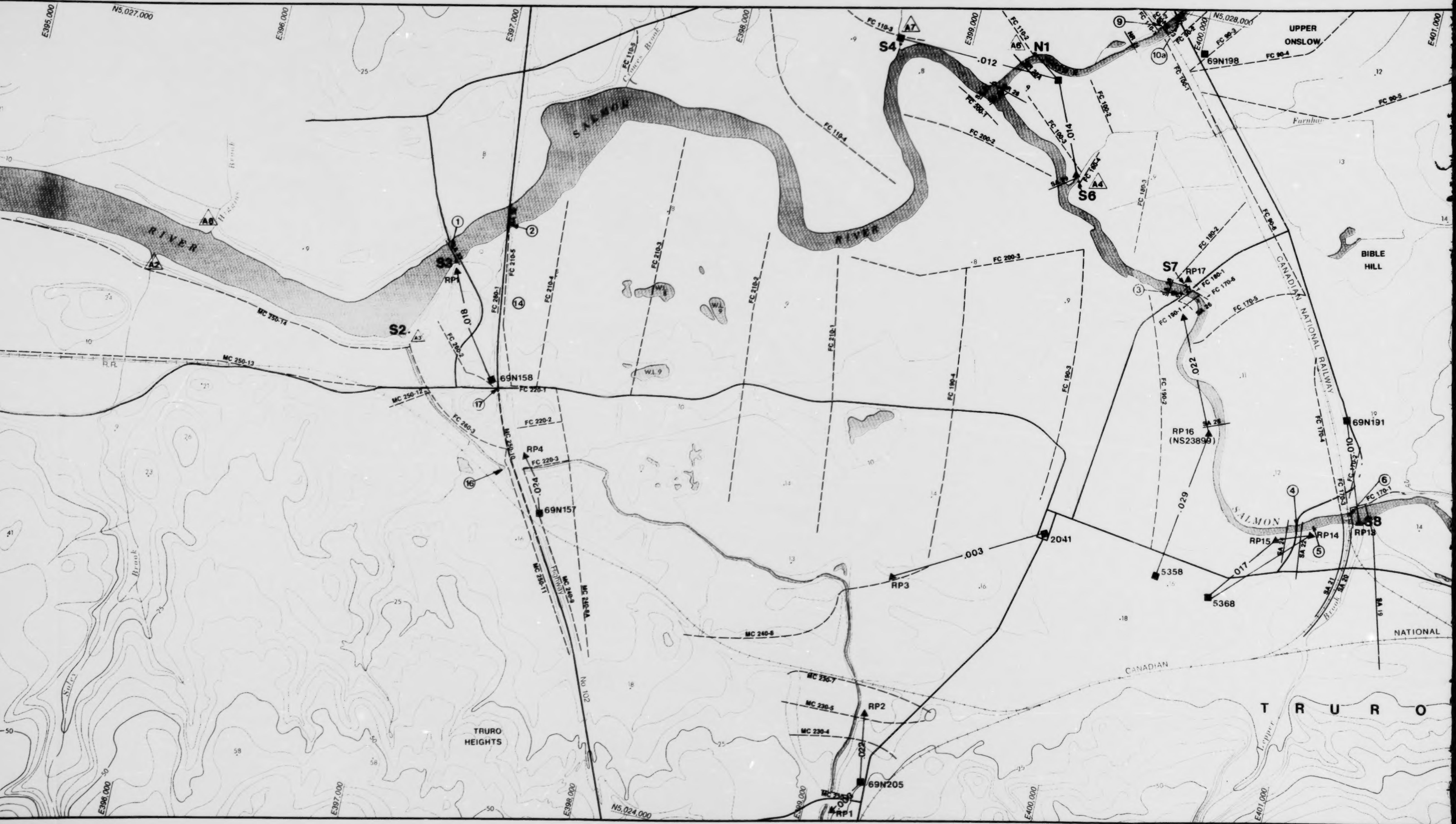
N5,026,000

FIGURE 5.1a



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UPPER ONSLOW
Farnham
BIBLE HILL
SALMON RIVER
RIVER
SALMON
Chiggins Brook
S2
S3
S4
S6
S7
S8
N1
A2
A3
A4
A6
A7
A8
RP1
RP2
RP3
RP4
RP13
RP14
RP15
RP16 (NS23899)
RP17
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FC 110-3
FC 110-4
FC 110-5
FC 200-2
FC 200-3
FC 210-2
FC 210-3
FC 210-4
FC 210-5
FC 200-1
FC 260-2
FC 260-3
FC 220-1
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FC 220-3
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FC 190-4
FC 180-1
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FC 180-3
FC 170-1
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FC 90-3
FC 90-4
FC 90-5
FC 90-6
MC 250-12
MC 250-13
MC 250-14
MC 240-8
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MC 230-4
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SA 19
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R.R.
CANADIAN NATIONAL RAILWAY
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No 102
TRURO HEIGHTS
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T R U R O
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E397,000
E398,000
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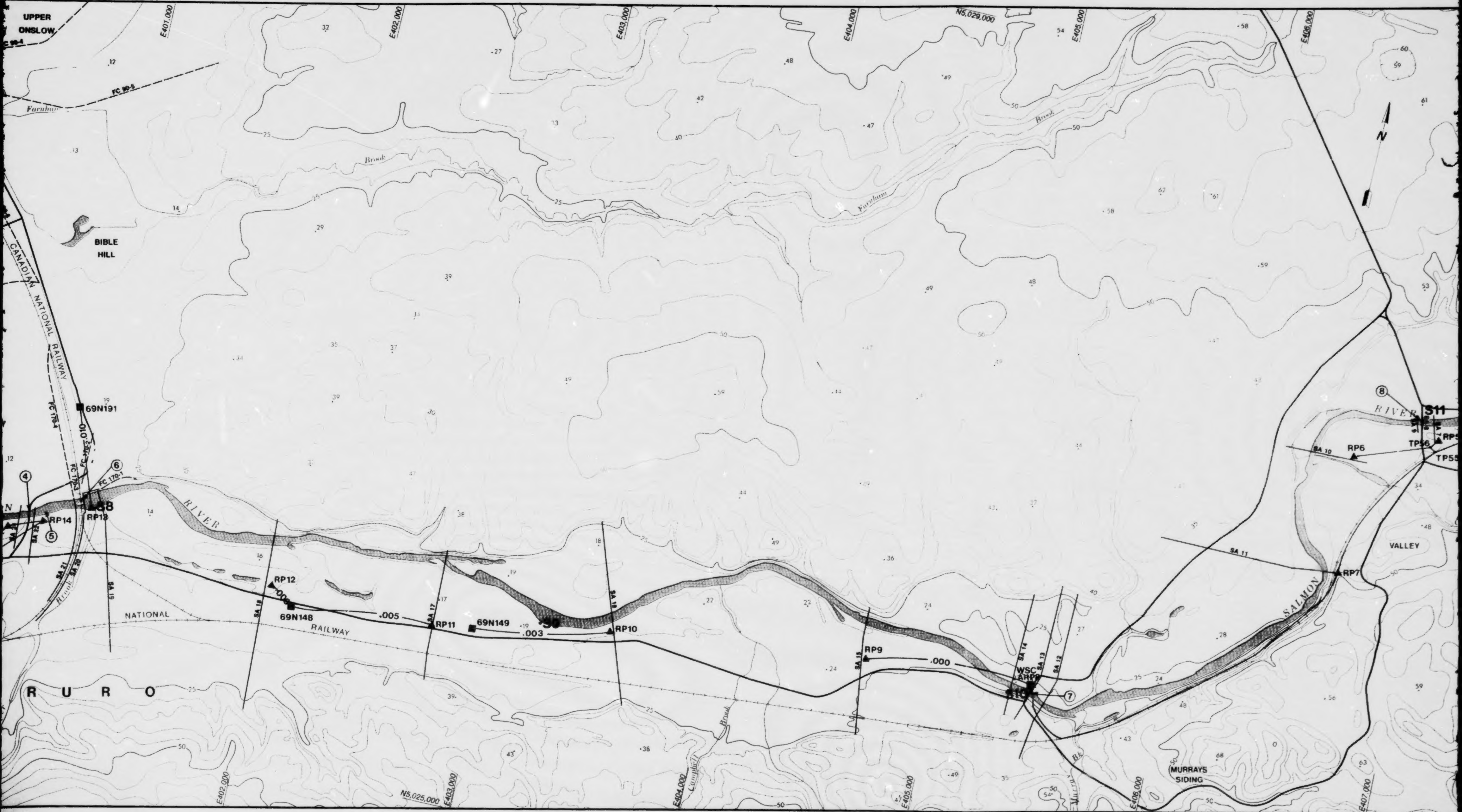
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SA 12
⑦
SALMON
RP7
SA 11
VALLEY
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RP6
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RIVER
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TP56
TP55
RP5
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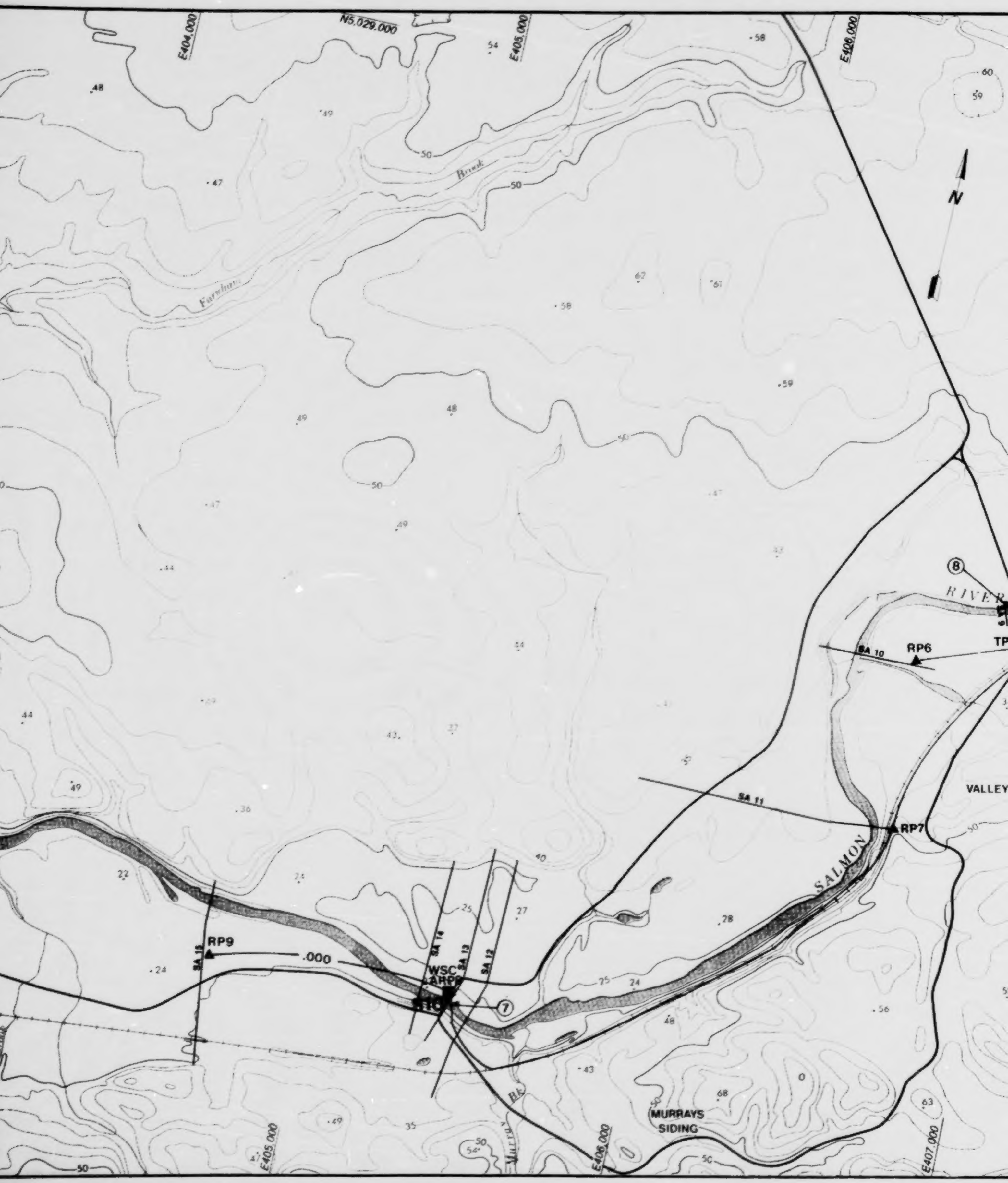
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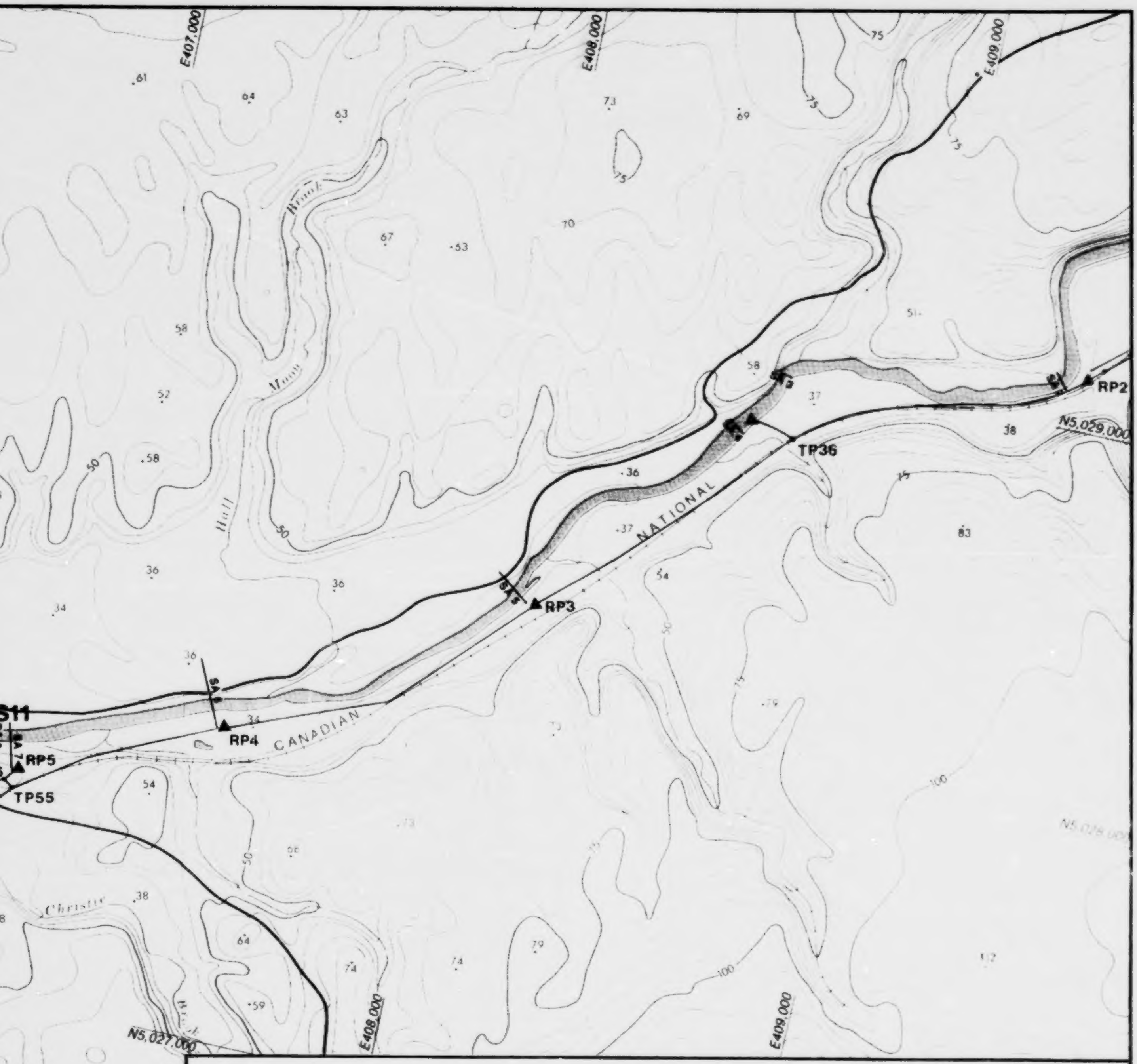
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SA 5
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CANADIAN
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Brook
Hall
Christie
50
75
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SCALE

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ÉCHELLE

CONTOUR INTERVAL 5 METRES

EQUIDISTANCE DES COURBES 5 MÈTRES

ALL ELEVATIONS ARE ABOVE
CANADIAN GEODETIC DATUM

LES ÉLEVATIONS SONT COTÉES
PAR RAPPORT AU RÉSEAU
GÉODÉSIQUE CANADIEN

LEGEND

.S6	STAFF GUAGE SITE
▲	LEVELS REFERENCE POINT
■—.000—▲	CIRCUT CLOSURE ERROR
①	HYDRAULIC STRUCTURE I.D. NUMBER
△A1	ABOITEAUX STRUCTURE I.D. NUMBER
—SA 1—	CROSS SECTION
—FC 170-7—	FLOOD CHANNEL
■	GEODETIC BENCH MARK

SALMON RIVER HYDRAULIC CHARACTERISTICS/ SURVEYS

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

TRURO HYDROTECHNICAL STUDY

6.0 HYDROLOGY

6.1 Hydrologic Description of Study Area

The Salmon and North Rivers, with headwaters in Colchester and Pictou counties, drain an area of about 726 km^2 into Cobequid Bay. The watersheds of the two rivers and their tributaries are shown in Figure 6.1. The North River, which is formed from three distinct branches, flows into the tidal portion of the Salmon River estuary at Upper Onslow. The Black River and Steele Run are the major tributaries to the Salmon River. There are several tributary brooks, principal ones being McClures, Lepper, Clifford, Christie, Farnham and McCurdy. The tributary drainage areas are listed in Table 6.1.

The catchment area is devoid of lakes and the land use is primarily forest cover with smaller amounts of agriculture and urban development. Reference to the profiles on Figure 6.2 will show that the rivers are relatively short with steep gradients, particularly in the upper reaches. Extensive channelization has been carried out in sections of the rivers, primarily at downstream locations, and this has altered the hydraulic characteristics over the years. Both river channels are silty near their confluence with Cobequid Bay, due to tide-borne sediment deposition, and are rocky in the upper reaches. The channel bed is characterized by numerous gravel bars in the downstream reaches of the freshwater portions of both rivers.

As a consequence of the lack of natural storage and the steep gradients, both river systems respond quickly to climatological and hydrologic events causing rapid fluctuations in discharge. High flows can occur at any time of the year; in the spring resulting from rapid snowmelt combined with rain; and/or in the fall as a result of heavy rains associated with major storms or hurricanes. Winter floods are also common from December to March due to rain on snow or frozen ground. High flows and ice jams have resulted in

TABLE 6.1

Drainage Areas

Tributary	(Area (km^2)
Salmon River at Valley	316.0
North River at North River	202.0
Christie Brook	30.6
Lepper Brook	19.6
Farnham Brook	20.2
McClures Brook	44.1
McCurdy Brook	20.5
Soley Brook	12.7
Higgins Brook	3.1
North River between North River and confluence with Salmon River	15.0
Small Drainage Areas up to downstream limit of Study Reach (Salmon River)	42.5
TOTAL DRAINAGE AREA	726.3 km^2

flooding at various times, as described in Chapter 3.0. Low flows occur in the summer months when the rivers can exhibit characteristics of small streams.

Discharge has been measured at three locations on the Salmon River for varying periods and at one location on the North River for a short time. In addition, Fraser Brook has been the subject of hydrological studies under the International Hydrologic Decade program and a hydrometric station is still in operation on the Brook near Archibald. The station locations are shown on Figure 6.1 and Table 6.2 summarizes the period of record of hydrometric observation.

The location and period of record of two meteorological stations in the Truro area are summarized in Table 6.2. Further detail on intensity, duration, frequency and characteristics of storms is given in Section 6.4.5.

The predominant soil types in the Salmon and North River basins have moderate runoff potential. Details on area soils and other factors influencing the runoff such as land use, antecedent soil moisture conditions, watershed slope, etc. are discussed in Section 6.4.

6.2 Overview of Hydrological Analyses

6.2.1 General

The basic objective of the hydrological analyses was to provide estimates of the 1:20 and 1:100 year design flood hydrographs at specified locations on the Salmon and North Rivers. The design flows were to be used for input to the hydraulic analysis for determination of water surface profiles in the flood prone areas of Truro and vicinity.

TABLE 6.2

Hydrometric and Meteorologic Data

A. W.S.C. Hydrometric Stations

Station Name	No.	D.A. (km^2)	Period of Record
1. Salmon River			
Union	01DH005	287.00	1977 - present
Valley	01DH001	316.00	1917 - 1926
Murray	01DH002	363.00	1964 - 1972
2. North River			
North River	01DH004	202.00	1976 - present
3. Fraser Brook			
Archibald	01DH003	9.07	1965 - present

B. Meteorological Stations

Station Name		Location Latitude	Location Longitude	Period of Record
1. Truro	8205990	45° 22'	63° 16'	1870 - 1915 1958 - present
2. Fraser Brook	8201850	45° 20'	63° 10'	1966 - 1978

Separate analysis approaches were used for the main streams and for the tributaries. The overall approach follows procedures recommended by the Regional Flood Frequency Analysis for Nova Scotia Streams (MacLaren Atlantic Ltd., 1980). The design flows on the Salmon and North Rivers were computed by application of regression equations (refer to Section 6.3.1). Design flows for the North River were required for its two branches, just upstream of their junction near Hydrometric Station No. 01DH004. The location of required flow for the Salmon River was at its confluence with Half Moon Brook. A design storm approach using rainfall-runoff modelling was adopted for the various locations along tributary streams (refer to Section 6.4), and in particular, at their confluence with the Salmon River at the locations shown in Figure 6.1.

In summary, flow estimates were made for the following locations:

1. by regression technique at:
    - Salmon River at Half Moon Brook
    - North River near North River
    - West Branch of the North River
    - South Branch of the North River

2. by rainfall-runoff method at:
    - mouth of Lepper Brook
    - mouth of Farnham Brook
    - mouth of McCurdy Brook
    - mouth of McClures Brook
    - mouth of Higgins Brook
    - mouth of Soley Brook
    - mouth of Christie Brook.

3. by pro-rating methods for lateral inflows along the Salmon and North Rivers for areas not considered in a) and b) above.

6.2.2 Methodology for Salmon and North Rivers

Peak flows were estimated using the regression technique for the Salmon and North Rivers. Design flow hydrograph shapes were then determined based on the average ratios of recorded peak flows and hourly flows as gauged at various locations on the main streams. Sensitivity testing of flow estimates was undertaken as described in Section 6.3.4 and the 95% confidence limits were also determined for the instantaneous peak flows based on the standard error of estimate procedure.

Finally, the flows for several historical floods were determined for the purpose of calibrating the hydraulic model as discussed in Section 6.3.6.

6.2.3 Methodology for Tributaries

The peak flows from the tributaries of the Salmon and North Rivers in the Truro area are affected by urbanization. Moreover, the drainage areas of many of the tributaries are at or below the minimum recommended for application of the regression equations developed by the regional method. (However, secondary regional comparisons were developed for Lepper, Farnham and McClures Brooks - see Section 6.4.7). Thus, regression equations could not be applied and the HYMO rainfall-runoff modelling procedures were adopted for providing tributary flow estimates (see Sections 6.4.1 and 6.4.2).

The HYMO model was calibrated and verified using relevant data for the Fraser Brook watershed (see Sections 6.4.3 and 6.4.4). The HYMO model was then applied to Christie, Farnham, Lepper, McCurdy, McClures, Higgins and Soley Brooks to determine the 1:20 and 1:100 year flows from corresponding design storms by determining appropriate basin parameters and soil information input, as described in Sections 6.4.4 and 6.4.5. The computed design flows were based on the 24-hour, 1:20 year and 1:100 year storms derived for data recorded at the Truro meteorological station. Estimates for the upper 95% confidence limit design flows were derived by inputting upper 95% confidence limit design storm data to the HYMO model and sensitivity analyses were undertaken as described in Section 6.4.7.

6.2.4 Lateral Inflows

Lateral inflows into the Salmon and North Rivers were calculated by pro-rating from the average design flood estimates derived from the various brooks close to each lateral area, as described in Section 6.4.8.

6.3 Flow Hydrographs - Salmon and North Rivers

6.3.1 Regional Hydrology

A Regional Flood Frequency Analysis for Nova Scotia was conducted under the Canada-Nova Scotia Flood Damage Reduction Program (MacLaren Atlantic, 1980).

This study undertook an analysis of watershed hydrologic, physiographic, and climatic characteristics and concluded that the mainland portion of Nova Scotia was a homogeneous region with respect to flood discharge characteristics. A regional coefficient of skew was estimated and the three parameter log normal distribution was applied to obtain single station estimates for the 1:2, 1:20 and 1:100 year flood peaks. Linear regression techniques were then utilized to develop functional prediction equations for the 1:2, 1:20 and 1:100 year mean daily flows as functions of watershed physiographic and climatic indices. The regional prediction equation for daily mean flood peak was found to take the following general form:

$$\ln QD_T = K + a \ln A + b \ln MAR + c \ln MSM + d \ln LONG + e \ln DF \qquad (6.1)$$

where QD_T = mean daily peak flow for recurrence interval T (T=1:2, 1:20, 1:100 years)
K,a,b,c,d,e = regression coefficients
A = drainage basin area (km^2)
MAR = index of mean annual runoff (mm)
MSM = index of median depth of snow on the ground at the end of March (mm)
LONG = degrees longitude of the basin outlet (nearest 5 seconds)
DF = drainage density factor, which is the number of permanent stream confluences divided by drainage basin area.

The reader is referred to Appendix B-1 for details of the application of this technique including parameter estimates, etc. for the Salmon and North Rivers.

A similar relationship for estimating the ratio of instantaneous to daily mean flood discharge rates was also developed in order to predict instantaneous discharge rates for ungauged watersheds. This equation takes the following form:

$$\ln \left(\frac{QP}{QD} - 1\right) = 234.771 - 0.454 \ln A - 0.187 \ln MSM - 6.929 \ln MAR - 44.803 \ln LONG \qquad (6.2)$$

where QP and QD are the peak and the mean daily flows.

The above relationships were applied in this study to estimate the 1:20 and 1:100 year recurrence interval instantaneous flood peaks. Guidelines were also provided in this study to identify locations where the flood generating characteristics of the watershed to the flood prone site may be different from those of the regional analysis. These areas are generally the ones with very small watershed area, where urbanization is occurring within the watershed, or where there is significant upstream regulation. McClures, Lepper and Farnham Brooks have been identified as small watersheds where urbanization effects influence the peak flows (MacLaren Atlantic Ltd., 1980). The study report, therefore, recommended the selection and use of a calibrated deterministic design storm modelling procedure for watersheds with drainage areas less than about 25 km^2. It was further concluded that single station analyses should not be applied where flow records are less than 25-35 years.

The regional analysis did not recommend any particular shape associated with peak flows. However, for the present investigation, several factors such as the physical configuration of the Salmon and North Rivers, the tidal effect, and influence of large storage areas along the study reach, dictated that hydrograph volumes as well as peak flows be determined. This necessitated defining the hydrograph shapes (volumes) associated with each design flood, as described in Section 6.3.3.

6.3.2 Computed Design Flood Flows

The watershed parameters which were determined for the Salmon River and North Rivers were found to fall within the range of applicability of the regional prediction equation (see Appendix B-1). The resulting instantaneous peak flow estimates are summarized in Table 6.3.

Design flood flows for the South and West Branch of the North River were computed by proportioning the estimated North River flow at North River by drainage area ratios of the two tributaries.

6.3.3 Design Flood Hydrographs

The tidal effects and the influence of large channel and floodplain storage areas along parts of the study reach dictated that hydrograph volumes and shapes associated with the peak flows be determined.

TABLE 6.3

Regional Flow Estimates

Stream Location	Daily Mean Flood Flow (QD) m^3/s		QP/QD	Instantaneous Flood Flow (QP) m^3/s			
	20 yr	100 yr		20 yr	95% C.L.	100 yr	95% C.L.
Salmon River at Valley	148.4	205.5	1.43	212.8	+70 -41	294.7	+82 -45
North River							
at North River	90.6	124.6	1.56	141.3	+82 -45	194.4	+94 -48
at Confluence with Salmon	94.4	129.1	1.54	145.3	+80 -45	198.8	+92 -48
West Branch North River above confluence with South Branch				48.75		67.07	
South Branch North River above confluence with West Branch				92.55		127.33	

The available discharge records on the North River at North River and the Salmon River at Valley and Union were analysed in order to obtain appropriate hydrograph shapes and volumes. It was found to be reasonable to assume that the North and Salmon Rivers peak at approximately the same time at their respective upstream study limits (refer to Appendix B-2).

The shapes of the 1:20 and 1:100 year design hydrographs were based on an analysis of measured hydrographs at the hydrometric gauge locations. Dimensionless hydrograph shapes were determined by dividing measured hydrograph ordinates by the peak flow of the hydrograph. This was repeated for several measured hydrographs and an average dimensionless hydrograph was determined as described in Appendix B. The 1:20 year and 1:100 year peak flows estimated from the regression equation were then applied to the dimensionless hydrograph resulting in the 1:20 and 1:100 year flood hydrographs for both rivers as shown on Figures 6.3, 6.4 and 6.5

6.3.4 Sensitivity Analysis

The sensitivity of peak flow estimates made by the regression method to changes in parameter values was evaluated by varying one parameter while holding the others at their initially determined values. Details of the sensitivity testing are given in Appendix B.

The flow estimates were found to be the most sensitive to changes in the following parameters; A, MAR and LONG. (Peak flow estimates were not sensitive to changes in the MSM and DF indices.) The index of longitude (LONG) can generally be derived without error. The other two of the more sensitive paramaters, A and MAR can generally be determined accurately from available information and variation of more than 10% are not likely. Variations of ±5% resulted in peak flow changes of +3 to -4 and -9 to +13% for A and MAR respectively.

The sensitivity testing indicated that variations in the peak flow estimates are within reasonable limits when the parameters are within the range on which the regional analysis was based. All variations were found to be well within the statistical confidence limits.

6.3.5 HYMO Model Application to Salmon and North Rivers

A general description of the HYMO model is given in Section 6.4.2 and a more detailed discussion of the present application is given in Appendix B.4. The HYMO model was applied to the Salmon and North River watersheds in order to obtain secondary peak flow estimates for comparison to the estimate by the regional method discussed above. The comparisons (see Appendix B.4) confirm the results of the flow estimate by the Regional Analysis and it was concluded that the flow estimates by the regional technique are reasonable and suitable for the purposes of this investigation.

6.3.6 Flows for Calibration and Validation of 1-D Model

Flow hydrographs were required for calibration and validation of the hydrodynamic model prior to simulating the flood profiles associated with the design flood hydrographs. Two significant flow events with recorded discharge data sufficient for calibration, occurred on April 16, 1980 and April 29, 1982. For calibration of the hydrodynamic model, data for these events were required for the main reaches of the Salmon and North Rivers and lateral areas along both rivers. For the April 29, 1982 event, the flow data had been recorded for the Salmon River at the Valley gauge whereas records are available for the Union gauge for the April 16, 1980 event (Table 6.4). For the North River, both events were recorded at the North River hydrometric station. Using the drainage area ratio, the North River flows were further proportioned for the West Branch and the South Branch above their junction with the main stream (Table 6.5).

TABLE 6.4

Recorded Flow Data for Salmon River

Salmon River at Valley 1982 Event		Salmon River at Union 1980 Event	
Date and Time (hr.)	Discharge (m^3/s)	Date and Time (hr.)	Discharge (m^3/s)
April 28 - 13	59.9	April 15 - 23	39.9
14	68.7	24	56.1
15	78.3	16 - 1	76.7
16	88.0	2	98.3
17	98.8	3	114.4
18	110.0	4	122.1
19	123.0	5	126.6
20	140.0	6	124.6
21	156.0	7	121.5
22	171.0	8	113.3
23	185.0	9	106.5
24	197.0	10	98.5
29 - 1	200.0	11	91.2
2	195.0	12	84.7
3	188.0	13	78.4
4	180.0	14	72.8
5	172.0	15	69.4
6	162.0	16	66.3
7	154.0	17	63.4
8	141.0	18	60.6
9	127.0	19	57.5
10	116.0	20	55.2
11	108.0	21	53.2
12	97.0	22	51.0
13	90.4	23	48.7
14	84.4	24	46.7

TABLE 6.5

Flow Data for North River at Stewart Bridge

1982 Event Discharges (m^3/s)				1980 Event Discharges (m^3/s)			
Date and Time (hr.)	North River	West* Branch	South* Branch	Date and Time (hr.)	North River	West* Branch	South* Branch
Apr. 28 - 16	135.0	44.5	85.1	Apr. 15 - 20	19.0	6.3	12.0
17	143.0	47.2	90.1	21	21.2	7.0	13.4
18	151.0	49.8	95.1	22	26.3	8.7	16.6
19	153.0	50.5	96.4	23	38.5	12.7	24.3
20	147.0	48.5	92.6	24	66.0	21.8	41.6
21	141.0	46.5	88.8	16 - 1	85.0	28.0	53.5
22	139.0	45.9	87.6	2	98.0	32.3	61.7
23	146.0	48.2	92.0	3	103.6	34.2	65.3
24	152.0	50.2	95.8	4	100.8	33.3	63.5
29 - 1	156.0	51.5	98.3	5	93.7	30.9	59.0
2	154.0	50.8	97.0	6	87.8	29.0	55.3
3	144.0	47.5	90.7	7	81.0	26.7	51.0
4	136.0	44.9	85.7	8	74.5	24.6	47.0
5	123.0	40.6	77.5	9	68.5	22.6	43.2
6	111.0	36.6	69.9	10	62.9	20.8	39.6
7	99.4	32.8	62.6	11	57.5	19.0	39.6
8	88.3	29.1	55.6	12	53.8	17.7	33.9
9	76.6	25.3	48.3	13	50.4	16.6	31.8
10	65.7	21.7	41.4	14	47.3	15.6	29.8
11	58.4	19.3	36.8	15	47.0	15.5	29.6
12	53.7	17.7	33.8	16	41.9	13.8	26.4
13	51.1	16.9	32.2	17	40.2	13.3	25.3
14	47.3	15.7	29.8	18	38.5	12.7	24.3
15	44.5	14.7	28.0	19	37.1	12.2	23.4
16	42.0	13.9	26.5	20	35.7	11.8	22.5
17	40.1	13.2	25.3	21	34.5	11.4	21.7
18	38.4	12.7	24.2	22	33.1	10.9	20.9
19	36.8	12.1	23.2	23	32.0	10.6	20.2
20	35.1	11.6	22.1	24	31.1	10.3	19.6

* By area pro-rating of North River measurement

To determine the additional inflows along both rivers to the downstream end of the study area, unit flows, expressed in cubic metre per square kilometre of the drainage area, were computed from the recorded flows at hydrometric stations near the boundary of the study area. The total additional inflows were then calculated based on the drainage area contributing flow within the study area, as given in Table 6.6.

6.3.7 Flow Data for 1971 Ice Event

It was also necessary to derive suitable inflows for simulation of an ice jam event by the hydrodynamic model. The February 1971 ice jam was selected as a typical event and corresponding distributed flow estimates were derived for input to the hydrodynamic model. Flow data for the February 1971 event were obtained by using available recorded flows for the Salmon River station at Murray. The hydrograph for the period February 8 to February 20, 1971 was constructed using daily flows and hourly stages. The peak flow was estimated considering the ratios of instantaneous to daily values, which were available for other periods. To obtain the inflow hydrograph at Valley, these values were simply prorated by area. As no data were available for the North River for 1971, the inflow hydrographs for the two North River Branches were obtained by prorating the Salmon River hydrograph with the respective drainage areas. A comparison of February flows for the North River station versus Salmon River flows for comparable time periods indicated this to be a reasonable approach.

Additional tributary and lateral inflows along the Salmon and North Rivers from the upstream boundary location to the area of interest were assumed to be equivalent to those determined previously for the 1:20 year event, since it was found that the magnitude of the February 1971 event corresponded to a peak flow with this recurrence interval.

TABLE 6.6

Inflows below Hydrometric Stations Events of 1980 and 1982

No.	Description of Area	Unit Flow (m^3/sec/Km^2)		Total Inflow (m^3/sec)	
		1980	1982	1980	1982
1	Salmon River				
	i) Salmon at Union Station to North River Junction	0.22	0.16	45.31	26.90
	ii) Downstream of Confluence with North River	0.22	0.16	19.82	13.87
2	North River From North River at Stewart Bridge to confluence with the Salmon River	0.27	0.14	5.66	3.68

6.4 Flow Hydrographs - Salmon and North River Tributaries

6.4.1 General

A number of small watersheds drain to the study area as indicated on Figure 6.6. Many of these small tributaries have outlets through aboiteaux built into the dykes in the study area. Unfortunately, there are no records of discharge measurements except for Fraser Brook, a watershed area of about 9.07 km^2, which contributes flows to the Salmon River as a sub-basin of Christie Brook. The Water Survey of Canada has collected discharge data on Fraser Brook from 1965 to date.

In general, the regional technique could not be used to obtain peak flow estimates for the tributary streams since some watershed characteristics were outside the range of application of the available equation.

In order to account for the effects of urbanization and other watershed characteristics, the HYMO model was selected for application. Further details of the model application are given in Section 6.4.2 and in Appendix B.

The HYMO model was calibrated and verified using recorded rainfall-runoff events for the Fraser Brook watershed as discussed in Sections 6.4.3 and 6.4.4. Information on model input parameters is given in Section 6.4.5 for the various tributaries. Finally, design flood hydrographs and sensitivity analyses were undertaken as discussed in Sections 6.4.6 and 6.4.7 respectively.

6.4.2 Description of the HYMO Model

The HYMO model is a single event watershed simulation model which was developed to determine the volume and peak flow of surface runoff. Rainfall data are transformed into runoff hydrographs and subsequently routed through stream valleys or reservoirs. Generally speaking, a watershed is broken down into sub-areas corresponding to the number of points at which flow estimates are required (see Figure 6.6). The runoff volume for each sub-area is computed by the U.S. Soil Conservation Service (SCS) Curve Number (CN) method which accounts for soil characteristics and antecedent moisture conditions. This volume is distributed in time using a unit hydrograph to obtain a hydrograph from each sub-watershed. The unit hydrograph is described by complex gamma and exponential decay functions of two parameters; namely the time to peak (t_p) and recession constant (k). A detailed description of the unit hydrograph is given in the user's manual (SCS, 1972).

When the model is applied to provide discharge estimates for an ungauged watershed, the appropriate values for the two parameters, time to peak (t_p) and recession constant (k), are most commonly estimated from empirical relationships which relate to major physiographic characteristics. Since the empirical relationships are not based on watersheds in Nova Scotia, it was considered to be desirable to calibrate the model in order to more accurately predict peak flows for the tributary watersheds.

Channel routing of flow hydrographs is accomplished using the Variable Travel Time method. This is a non-linear procedure which adjusts travel time through a reach to account for the increasing velocity of flow as the depth of flow increases. In order to utilize this method the slope and length of the reach must be specified with a 'typical' valley channel cross-section; the only parameters which must be estimated are values of Manning's 'n' for the floodplain and for the channel.

In summary, the model is easy to use, flexible and requires input of only three parameters (t_p, k and CN) per sub-area and four parameters (valley cross-section, Manning's 'n', slope and length) per channel routing section.

6.4.3 Fraser Brook Model Calibration

In the absence of adequate historical data for small watersheds, it is reasonable to calibrate the HYMO model using available records in a similar watershed, check its validity and then use the results of this analysis to obtain parameters and flow estimates for the ungauged watersheds of interest. The HYMO model was set up for the Fraser Brook watershed which is within the study area and thus soils, precipitation and runoff characteristics are generally representative of the adjoining watersheds.

Precipitation data for the Truro area is available at the Truro Station from 1958 to the present and at the Fraser Brook IHD station for the period 1966-1978. For the concurrent period of hourly precipitation at Truro and discharge records at Fraser Brook, two major events were identified, August 1971 (Hurricane Beth), and February 1971.

The August 1971 data were used for model calibration and the other events for verification. The rainfall amounts associated with the storm of August 1971 were recorded as 139 mm at the Fraser Brook station and 131 mm at the Truro station. The physical model input parameters are discussed in Appendix B.3.1. The resulting calibration simulation is shown on Figure 6.7, which indicates a very good agreement between simulated and measured peak flows and volumes.

6.4.4 Model Verification

An attempt was made to verify the HYMO model using the February 1971 event. The calibrated values of k and t_p were used in the simulations and were found to adequately represent the timing characteristics of the unit hydrographs (see Figure 6.8). However, it was found to be necessary to adjust the average antecedent soil moisture index to reflect actual conditions prior to the events (see Appendix B.3.2). However, the recession side of the measured and simulated hydrographs are somewhat different. This is attributed to the occurrence of snowmelt over the 3 day period, February 12, 13 and 14 and explains the difference in measured and simulated hydrographs. Since this snowmelt volume is not included in the HYMO simulation, the model is considered to be adequate for simulation of rainfall-runoff conditions.

6.4.5 HYMO Model Input

i) Design Storms

It was necessary to input the 1:20 and 1:100 year design storms in order to estimate the corresponding peak flows. The most appropriate meteorological station was determined to be located at Truro with a good long-term record from 1870 to 1915 and 1958 to date. (Additional supporting information regarding the derivation of design storm characteristics is given in Appendix B.) The rainfall amounts for various durations were analysed to determine critical runoff volumes and peak flows (see Appendix B.3.4). It was concluded that the 24-hour duration would result in the most critical conditions with respect to the hydrodynamic simulation of flood profiles. The total rainfall amounts of 102.7 and 129.0 mm correspond to the 1:20 and 1:100 year rainfall amounts respectively. The rainfall totals were then distributed in 30-minute time steps over the 24-hour period using the average AES temporal distribution (see Appendix B.3.4). The resulting 24-hour design storm inputs for the HYMO model are summarized in Table 6.7 for the mean rainfall and upper 95% confidence limits.

ii) Physical Parameters

The watershed areas of the seven tributary streams are shown on Figure 6.6. The sub-basins shown for each watershed were delineated on the basis of drainage patterns, type of cover and land use (i.e. urban, rural and forest cover, etc.).

TABLE 6.7

24-Hour Design Storm Input for HYMO Model

(Mean and 95% C.L.)*

Time from Start of Storm (min)	Accumulated Rainfall (mm) 1:20-Year Design Storm Mean	 +95% C.L.	 1:100-Year Design Storm Mean	 +95% C.L.
0	0	0	0	0
30	0.50	0.51	0.75	0.76
60	1.02	1.27	1.27	1.52
90	1.52	2.03	2.03	2.54
120	2.54	3.05	3.30	4.06
150	3.30	4.06	4.06	5.33
180	5.08	6.35	6.35	8.38
210	6.10	7.62	7.62	9.91
240	7.11	8.89	8.89	11.43
270	9.14	11.43	11.68	14.73
300	11.18	13.97	14.22	18.03
330	12.70	15.75	16.00	20.57
360	15.24	19.05	19.30	24.64
390	17.53	21.59	21.84	28.19
420	20.57	25.40	25.91	33.02
450	23.62	29.21	29.72	38.10
480	26.67	33.02	33.53	42.93
510	29.72	36.83	37.34	48.01
540	34.80	43.18	43.69	56.13
570	39.37	48.77	49.53	63.75
600	42.93	53.34	54.10	69.60
630	44.96	55.88	56.64	72.90
660	49.28	60.96	61.98	79.50
690	52.83	65.28	66.55	85.34
720	55.88	69.09	70.36	90.17
750	58.93	72.90	74.17	95.25
780	61.98	76.71	77.98	100.08
810	65.02	80.77	81.79	105.16

TABLE 6.7 (cont'd)

Time from Start of Storm (min)	Accumulated Rainfall (mm) 1:20-Year Design Storm Mean	+95% C.L.	1:100-Year Design Storm Mean	+95% C.L.
840	68.07	84.33	85.85	109.98
870	70.61	87.63	88.90	114.30
900	72.90	90.17	91.44	117.60
930	76.45	94.49	96.01	123.44
960	77.98	96.52	98.04	125.73
990	81.03	100.33	101.85	130.81
1020	84.07	104.14	105.66	135.64
1050	85.09	105.41	106.93	137.41
1080	87.12	107.95	109.73	140.72
1110	89.15	110.49	112.27	144.02
1140	92.20	114.30	116.08	149.10
1170	92.71	114.81	116.84	149.86
1200	94.23	116.84	118.62	152.40
1230	95.76	118.62	120.40	154.69
1260	97.54	120.65	122.43	157.23
1290	98.55	121.92	123.95	159.00
1320	99.97	123.19	125.22	160.53
1350	101.09	124.97	127.00	163.07
1380	101.60	125.73	127.76	163.83
1410	102.11	126.24	128.27	164.85
1440	102.70	126.95	129.00	165.53

* C.L. = +95% Confidence Limits

The watershed boundaries and physical characteristics were derived from available mapping. Topographic mapping with one metre contour intervals was available for the lower portions of tributary streams within the study area. For the upper watershed areas, maps with 5-metre contour intervals were obtained from the Land Registration and Information Service. In some instances 1:50,000 National Topographic Series maps were used for upper drainage basin areas. The physical parameters for HYMO model input such as length of stream, basin elevation difference, drainage area and valley cross-sections were based on the 5-metre contour interval topographic maps. The physical parameters are listed in Table 6.8.

In the case of McCurdy and Higgins Brooks, sub-areas were not considered to be necessary since watershed and channel characteristics were uniform. However, for the other brooks, each watershed area was divided into smaller sub-basins due to differences in urban and rural drainage land-use patterns. In these cases, the flow hydrograph from the upper sub-area was routed through the lower sub-area by using rating curves developed by the HYMO model based on the cross-section input data. The latter was obtained from the available topographic mapping. The hydrograph computed as inflow from the lower sub-area was then added onto the routed hydrograph to obtain the design flow at the outlet of the lower sub-area and this process was continued to the outlet.

iii) Soil Cover Complex Number

The HYMO model uses a runoff index factor which combines the soil group and land use characteristics and is referred to as a hydrologic soil cover complex number (CN). The soils for each sub-basin in each of the seven tributary watersheds were classified by hydrologic soil group using the soil characteristics according to the procedures discussed in Appendix B.3.3. Information on soil characteristics such as slope, stoniness, drainage, rockiness, soil profile and soil association name was obtained from the Soils and Crop Branch of the Nova Scotia Department of Agriculture, including

TABLE 6.8

Physical Parameters for Tributaries

Name of Brook and Sub-basin	Drainage Area A (km^2)	Stream Length L (km)	Elevation Difference H (metres)	Weighted Curve Number CN (AMC II)	CN (AMC III)
1. Christie Brook					
Sub-area 1 (Fraser Bk)	9.07	3.70	79.25	69.0	82.8
Sub-area 2	21.48	6.92	73.15	67.0	81.0
2. Lepper Brook					
Sub-area 1	16.97	8.69	106.68	65.5	82.0
Sub-area 2	2.67	3.38	65.53	79.2	91.0
3. Farnham Brook					
Sub-area 1	8.52	4.83	82.29	55.6	75.0
Sub-area 2	4.68	5.79	32.00	76.9	89.0
Sub-areas 3, 4 & 5	7.00	5.21	53.34	50.6	69.0
4. McCurdy Brook	20.50	9.66	137.16	71.0	85.0
5. McClures Brook					
Sub-area 1	40.43	10.67	115.82	79.9	86.0
Sub-area 2	2.90	1.71	11.58	82.2	92.0
Sub-area 3	0.77	5.21	7.62	79.7	91.0
6. Soley Brook					
Sub-area 1	10.62	7.56	128.02	72.0	86.0
Sub-area 2	2.08	1.93	54.86	77.0	89.0
7. Higgins Brook	3.10	2.25	27.43	76.0	89.0

surficial soil maps of the Truro area. The soil classification and land use characteristics for each sub-area (see Appendix B) were used to assign complex numbers according to procedure described in the U.S. Soil Conservation Service Handbook (SCS, 1972) (also see Appendix B). The weighted complex numbers for Antecedent Moisture Conditions II and III (AMC II and AMC III are summarized in Table 6.8). Antecedent moisture condition II represents the case where soil saturation conditions are average prior to the design flood event and condition III represents the situation where significant rainfall occurring prior to the event has saturated the soil and/or the ground may be frozen or partially frozen. For example, saturated condition CN (III) values were used for the February 1971 verification event for Fraser Brook.

6.4.6 Computed Design Flood Hydrographs

The 1:20 and 1:100 year design flood hydrographs for the tributaries were computed by the HYMO model with the following inputs:

1. 24-hour storm rainfall distributed at 30-minute intervals as per Truro rainfall statistics and the average A.E.S. temporal distribution (Table 6.7)
2. The watershed physical parameters such as drainage area, stream length and elevation difference from head reach to the required location of design flood hydrograph (Table 6.8)
3. The runoff index factor in the form of curve numbers (CN) as given in Table 6.8
4. Valley cross-sections taken from existing available mapping.

The calibrated unit hydrograph values (k and t_p) were used in the assessment of design flows for Christie Brook in sub-area 1 shown in Figure 6.6. However, for the other watersheds, local estimates of k and t_p were required since the basin characteristics were different from those of Fraser Brook. Therefore, the elevation difference H, and stream length L, were used as inputs to the HYMO model which then calculated k and t_p internally using empirical relationships.

The computed design flow hydrographs for tributary brooks are shown on Figures 6.9 to 6.15 and the corresponding peak flows are summarized in Table 6.9.

6.4.7 Sensitivity Analysis

Possible variations in peak flows were assessed in two ways, namely:

1. By inputting rainfall associated with the upper 95% rainfall amounts
2. By assuming saturated soil moisture conditions rather than average antecedent soil moisture conditions.

For the upper 95 percent confidence limit design flow hydrograph computations, the rainfall totals were adjusted upward, according to the incremental confidence limits given in Table 6.7. The resulting rainfall totals for the upper 95% rainstorm were determined to be 126.95 mm and 165.53 mm for the 1:20 and 1:100 year storms respectively. The computed design hydrographs for 95 percent confidence limits are shown on Figures 6.9 to 6.15.

Design flood hydrographs were also determined for both the average CN II condition and also the saturated CN III condition. These hydrographs are also shown in Figures 6.9 to 6.15. With reference to the resulting hydrographs, it is evident that changes to antecedent soil moisture result in larger differences in flow estimates compared to changes to the input rainfall according to the upper 95% confidence limit.

A comparison of peak flows for average and saturated conditions to that of the values determined by Regional Analysis (see Section 6.3.1) is also given in Table 6.9. The regional analysis figures are available for only three brooks in the area, and a single station estimate using available Fraser Brook data is also presented. The regional frequency analysis had confidence limits associated

TABLE 6.9

Comparison of CN (AMC II, AMC III), and Regional Analysis Peak Flows for Various Brooks

No.	Name of Brook	Drainage Area (km^2)	20-year Design Flows (m^3/s)			100-year Design Flows (m^3/s)		
			Regional Analysis	CN CN (II)	CN CN (III)	Regional Analysis	CN CN (II)	CN CN (III)
1.	Christie Brook	30.55	-	16.93	28.88	-	26.28	40.15
	Fraser Brook	9.07	(7.36)	4.95	8.24	(9.39)	7.56	11.35
2.	Lepper Brook	19.64	32.30	11.41	20.05	43.70	17.50	27.49
3.	McClures Brook	44.10	59.40	31.29	51.28	78.70	46.24	69.46
4.	Farnham Brook	20.20	34.90	8.55	18.75	47.70	14.30	26.70
5.	McCurdy Brook	20.50	-	15.23	24.04	-	22.45	33.07
6.	Higgins Brook	3.10	-	3.00	4.30	-	4.22	5.80
7.	Soley Brook	12.70	-	10.79	16.28	-	15.57	22.11

NOTE: ( ) Figures in brackets are based on single station analysis (See Appendix B) with the peak flow estimates on the order of +40% to -70%. Hence higher peaks then summarized for the Regional Analysis could reasonably be expected. A comparison indicates that CN II peak flow values are lower in magnitude than the Regional Analysis and that CN III, saturated conditions are generally closer to the Regional Analysis. In view of the high variability associated with flow estimates for small watersheds, the hydrographs associated with the mean rainfall and saturated soil conditions were considered to be the most appropriate for input to the hydrodynamic model.

6.4.8 Lateral Inflows

i) Salmon River

The shoreline area of the Salmon River along the river reach between Valley Station and Cobequid Bay contributes to flood flows directly either in the form of overland flow or through small creeks. A total of 42.5 km^2 drains both sides of the river, in addition to the tributaries previously discussed, and can be characterized according to three sub-areas as follows:

1. area between upstream end of study reach to Lepper Brook without urban development influence
2. area between Lepper Brook and McClures Brook influenced by urban developments
3. area between McCurdy Brook and downstream end of study reach without urban development.

The three lateral flow areas are shown in Figure 6.6. Design flood flows for these three areas were based on prorating the HYMO model peak flows computed for adjacent brooks. Thus in the first reach, unit runoff from Sub-area 2 of the Christie Brook sub-basin was applied to calculate the total runoff for the 25.9 km^2 watershed area as indicated on Table 6.10.

TABLE 6.10
Lateral Inflows from Shoreline Areas Along Salmon River and North River

No.	Description of Lateral Area	Drainage Area (km²)	Design Flows (m³/s) Mean Instantaneous 20 Yr.	Mean Instantaneous 100 Yr.	+95% C.L. Flow 20 Yr.	+95% C.L. Flow 100 Yr.
1	Upstream Study Reach to Lepper Brook - based on Christie Brook (subarea 2)	25.9	7.77	11.03	10.76	15.73
2	Lepper Brook to McClures Brook - based on Farnham Bk (sub-areas 3,4 & 5) and McClures Brook (subarea 3)	5.9	7.85	11.01	10.76	15.63
3	McCurdy Brook to downstream end of study reach - based on McCurdy and Soley Bks.	10.7	6.96	9.48	9.29	13.07
4	North River Reach from study area boundary to confluence with Salmon River - based on Farnham Brook sub-areas 3,4 & 5) and McCurdy Brook	15.0	15.31	22.06	21.53	32.02

Similarly for the second shoreline area, Farnham Brook (sub-areas 3, 4 and 5) and McClures Brook (sub-area 3) were used. For the third area flows were based on McCurdy and Soley Brooks runoff characteristics as shown on Table 6.10. The lateral flows from reach 2 between Lepper Brook to McClures Brook are higher in comparison to the other two reaches due to urban effects.

ii) North River

Lateral inflows from shoreline areas along the North River were calculated using the same approach as the Salmon River. An area of about 15 km^2 along the North River contributes flows directly into the river. This reach extends from the confluence of the South and West Branch to the confluence of the North River with the Salmon River. The shoreline areas are adjacent to Farnham Brook sub-areas 3, 4 and 5 (Figure 6.6) and McCurdy Brook, and, therefore, flows were based on the average unit runoff from these Brooks. The calculated mean daily and 95% confidence limit flows for the 1:20 year and 1:100 year peaks (CN III) are also presented in Table 6.10.

6.5 Summary and Conclusions

6.5.1 Summary of Methodology

The 1:20 and 1:100 year design flows for the Salmon and North Rivers were determined using the regression equations and procedures recommended in the Regional Flood Frequency Analysis for mainland Nova Scotia streams. A sensitivity analysis of variations in design flows was undertaken by varying the significant input parameters such as drainage basin area, index of mean annual runoff, index of median depth of snow on the ground at the end of March, the drainage density factor and the longitude of drainage basin outlet. An analysis of the timing of flow peaks for both rivers was undertaken to determine the accumulated effect of flows input to the hydrodynamic model. The design flow hydrograph shapes for both rivers were based on observed hydrographs for the available period of record. The average hydrograph of the recorded flows was selected to represent the volume of runoff and a sequence of flow magnitudes during the flood. The recorded hydrographs during the winter also included the influence of snowmelt or ice conditions.

The design flows for various brooks contributing flows to the Salmon River within the study reach were computed by using the HYMO model. The model was calibrated to the recorded August 1971 Hurricane Beth event for Fraser Brook and then validated with the February 1971 event. The HYMO model was then applied to Christie, Farnham, Lepper, McCurdy, McClures, Soley and Higgins Brooks for computation of 1:20 and 1:100 year design floods. Peak flows were also determined by varying the antecedent moisture conditions and by determining the upper 95 percent confidence limit flows. The lateral inflows from shoreline areas along the Salmon and North Rivers were based on unit runoff computed by the HYMO model for watersheds of brooks adjacent to the area.

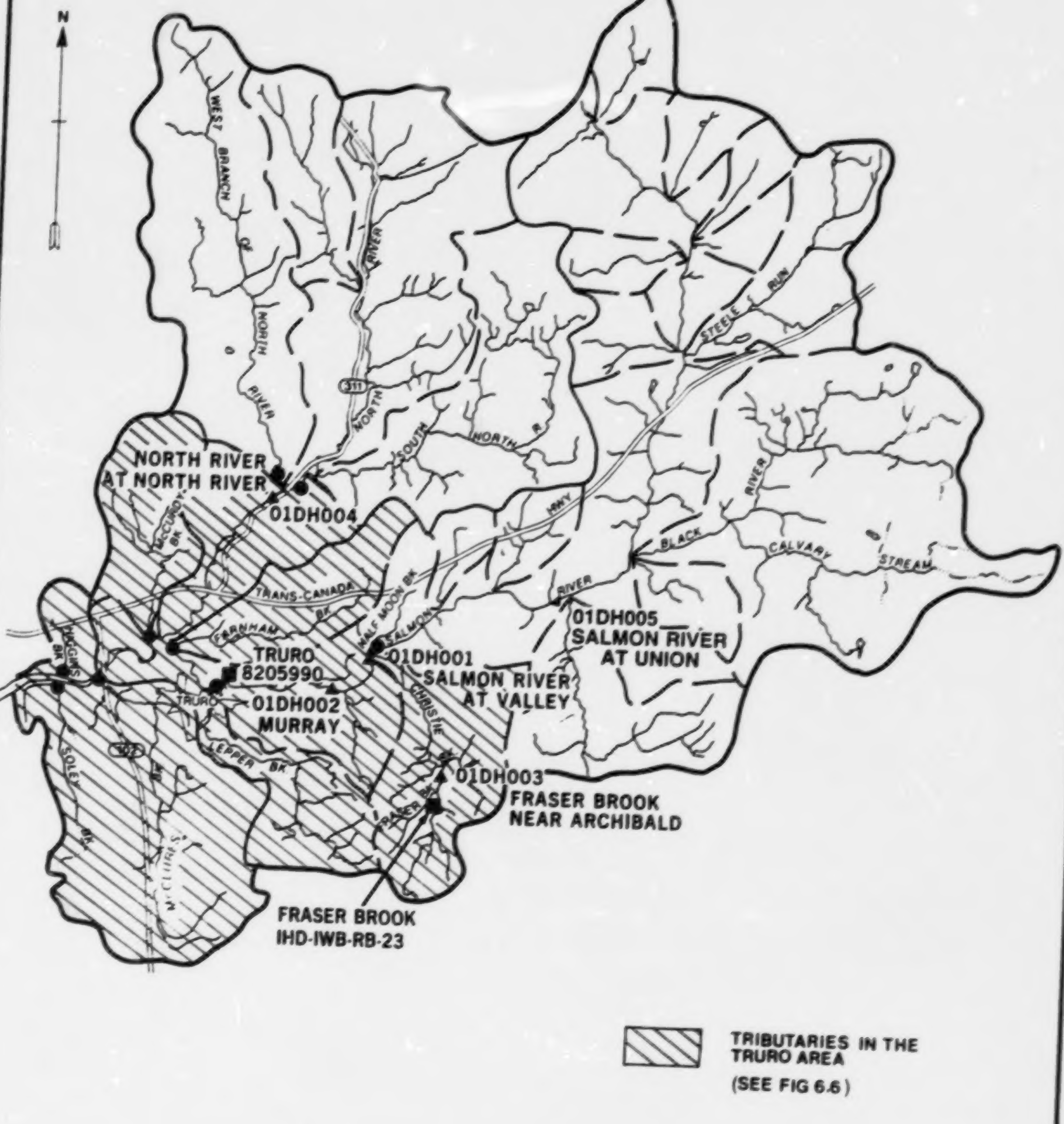
6.5.2 Conclusions

1. The analysis of the timing of peak flows (based on observed data for the Salmon and North Rivers) indicates that the two rivers generally peak at the same time. The maximum recorded variation for high flow events was two hours, with the North River at Stewart Bridge peaking earlier than the Salmon River at Union. Due to this small difference, for the purpose of hydrodynamic modelling, the two rivers are assumed to peak at the same time. However, it was found that the brooks in the study reach normally peak earlier (4 to 9 hours). For the purposes of this investigation, the start of the hydrograph rise from baseflow for the two rivers and the tributaries was assumed to be coincident. As reported in Section 8.6.4, the relative timing of tributary and lateral inflows in relation to flows in the main river has little effect on flood stage due to their relatively small magnitude.

2. The sensitivity analyses carried out to show the effect of parameter changes on the design flows of the Salmon and North Rivers indicated that all variations are well within the expected statistical confidence limit as recommended in the regional flood frequency analysis.

3. The calibration and verification runs for the HYMO model support the use of this technique for estimation of design flows for the various tributary streams.

4. A comparison of peak flows estimated by the regional analysis with the flows estimated by the HYMO model using the average (AMC II) and saturated (AMC III) antecedent moisture conditions indicates that the flow estimates associated with saturated soil moisture conditions are more comparable to the regional values. Flow estimates associated with AMC II are on the low side. The design flows with saturated conditions also partially reflect the effect of snowmelt on antecedent moisture conditions as validated by the HYMO model verification run. Consequently, it is recommended that peak flows associated with saturated conditions (AMC III) be used for input to the hydrodynamic model for the tributaries and lateral drainage areas.


N
WEST BRANCH
NORTH RIVER
311
NORTH RIVER
NORTH
SOUTH
NORTH R.
STEELE RUN
NORTH RIVER
AT NORTH RIVER
01DH004
McCURDY BK
TRANS-CANADA
HWY
FARNHAM BK
HALF MOON BK
SALMON
TRURO
8205990
01DH001
SALMON RIVER
AT VALLEY
RIVER
01DH005
SALMON RIVER
AT UNION
BLACK RIVER
CALVARY STREAM
TRURO
01DH002
MURRAY
HIGGINS BK
SOLEY BK
102
McCLURES BK
LEPPER BK
CHRISTIE BK
01DH003
FRASER BROOK
NEAR ARCHIBALD
FRASER BK
FRASER BROOK
IHD-IWB-RB-23
TRIBUTARIES IN THE TRURO AREA
(SEE FIG 6.6)

● REQUIRED DESIGN FLOOD HYDROGRAPH LOCATION

▲ WATER SURVEY OF CANADA GAUGE

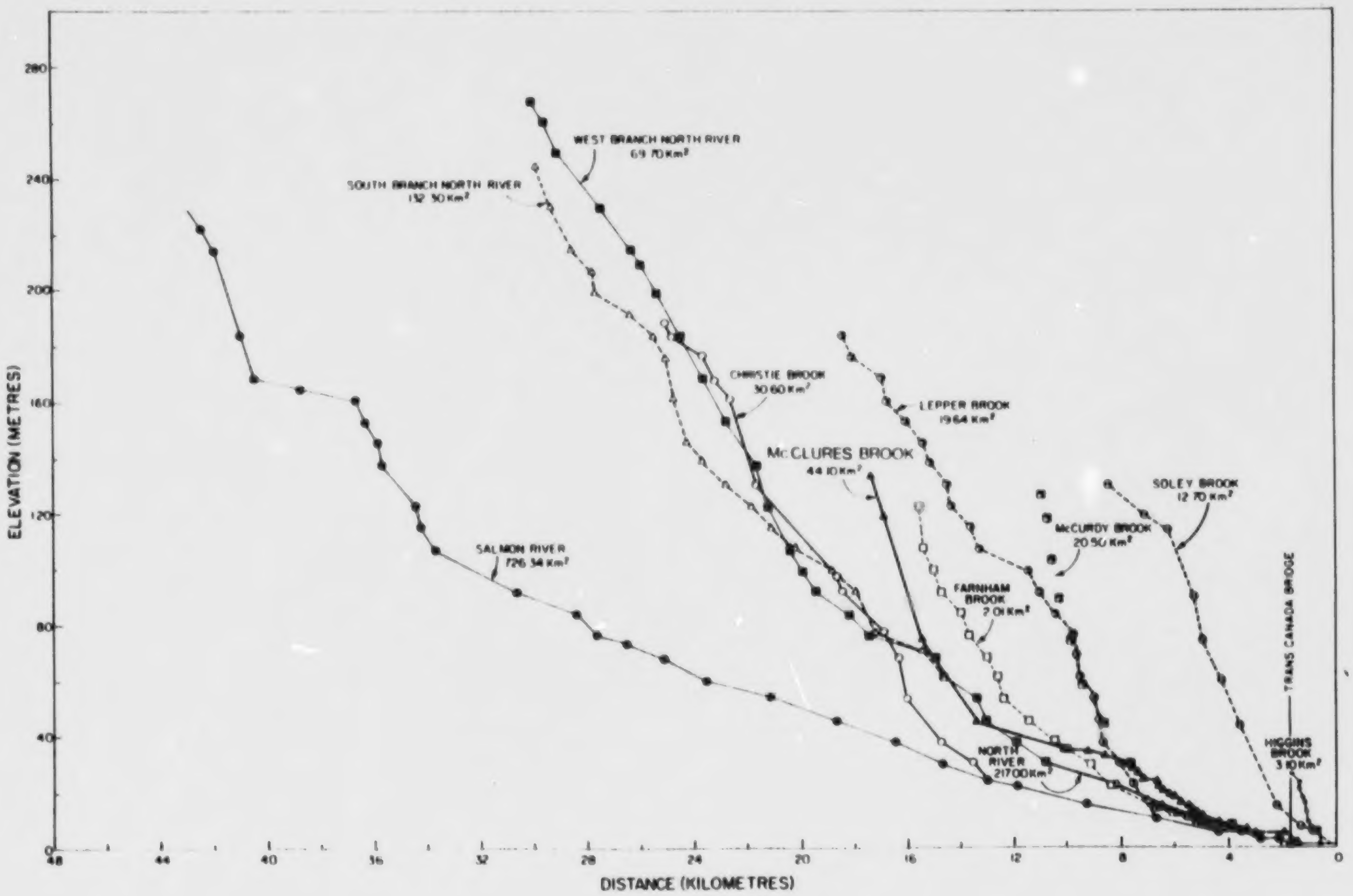
■ METEOROLOGICAL STATION

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

SALMON RIVER WATERSHED

1 0 1 2 3 4 KILOMETRES

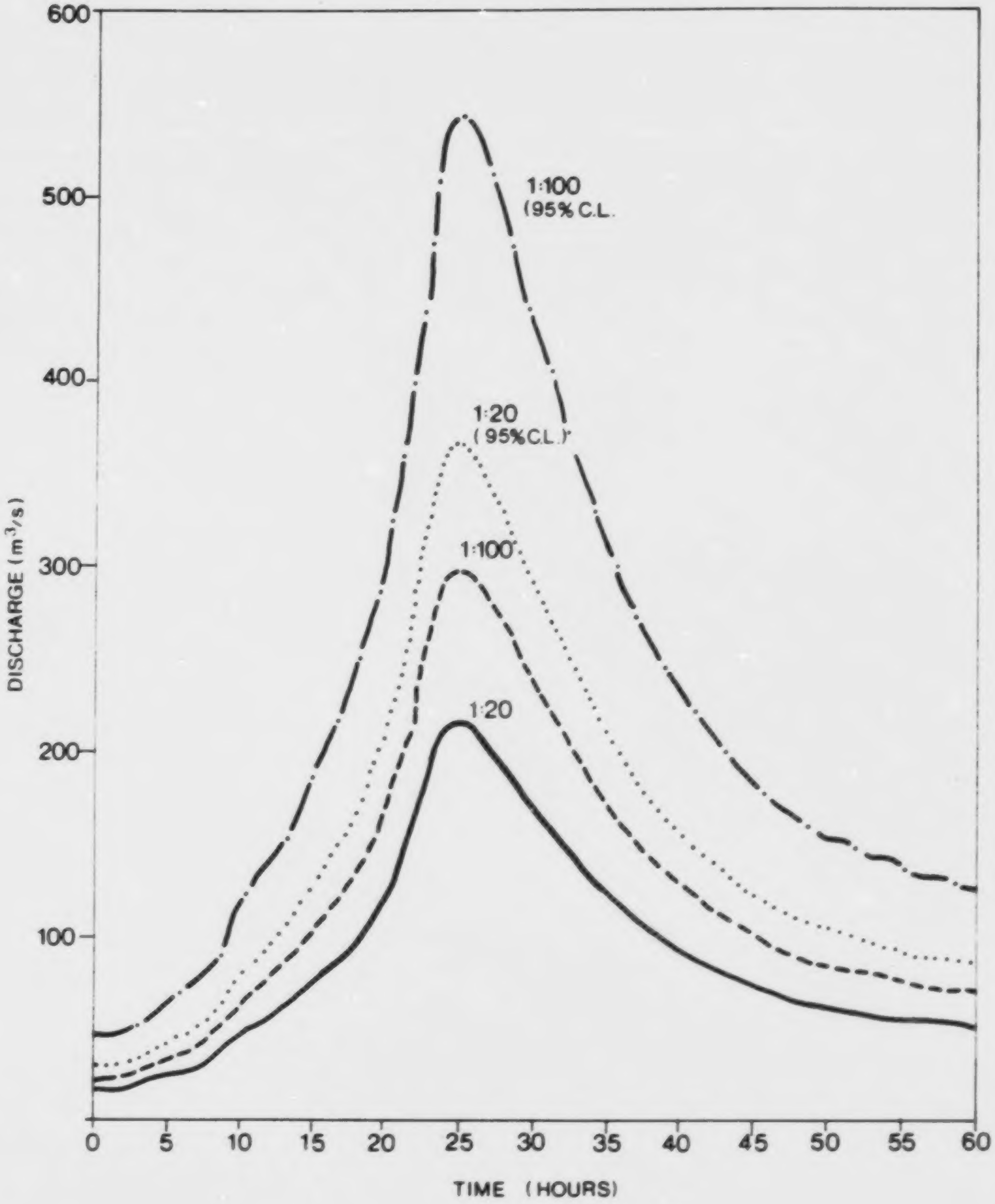
FIGURE 6.1



CANADA - NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

RIVER PROFILES - SALMON AND NORTH RIVER SYSTEMS

FIGURE 6.2



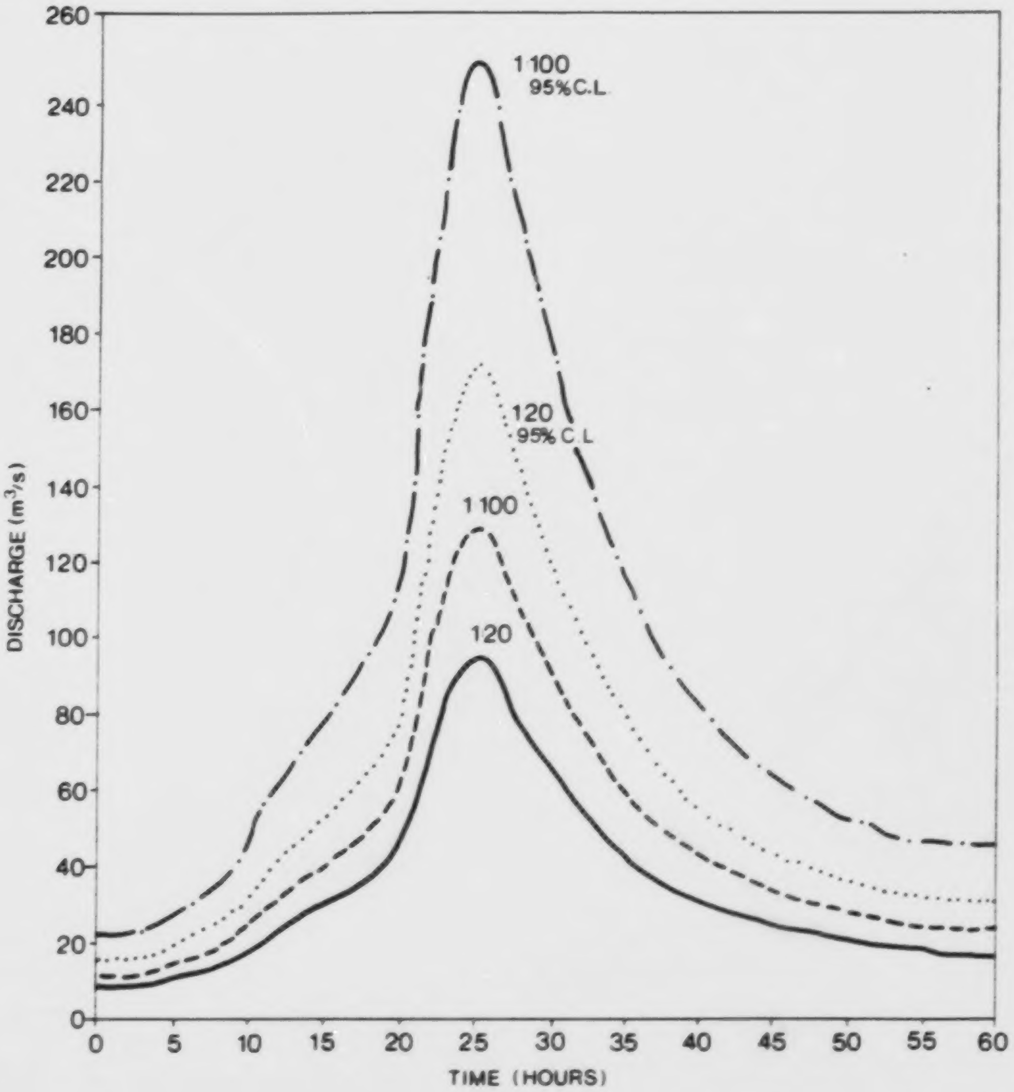
CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

TRURO HYDROTECHNICAL STUDY

DESIGN FLOOD HYDROGRAPHS FOR SALMON RIVER AT VALLEY

STN. IDH001

FIGURE 6.3



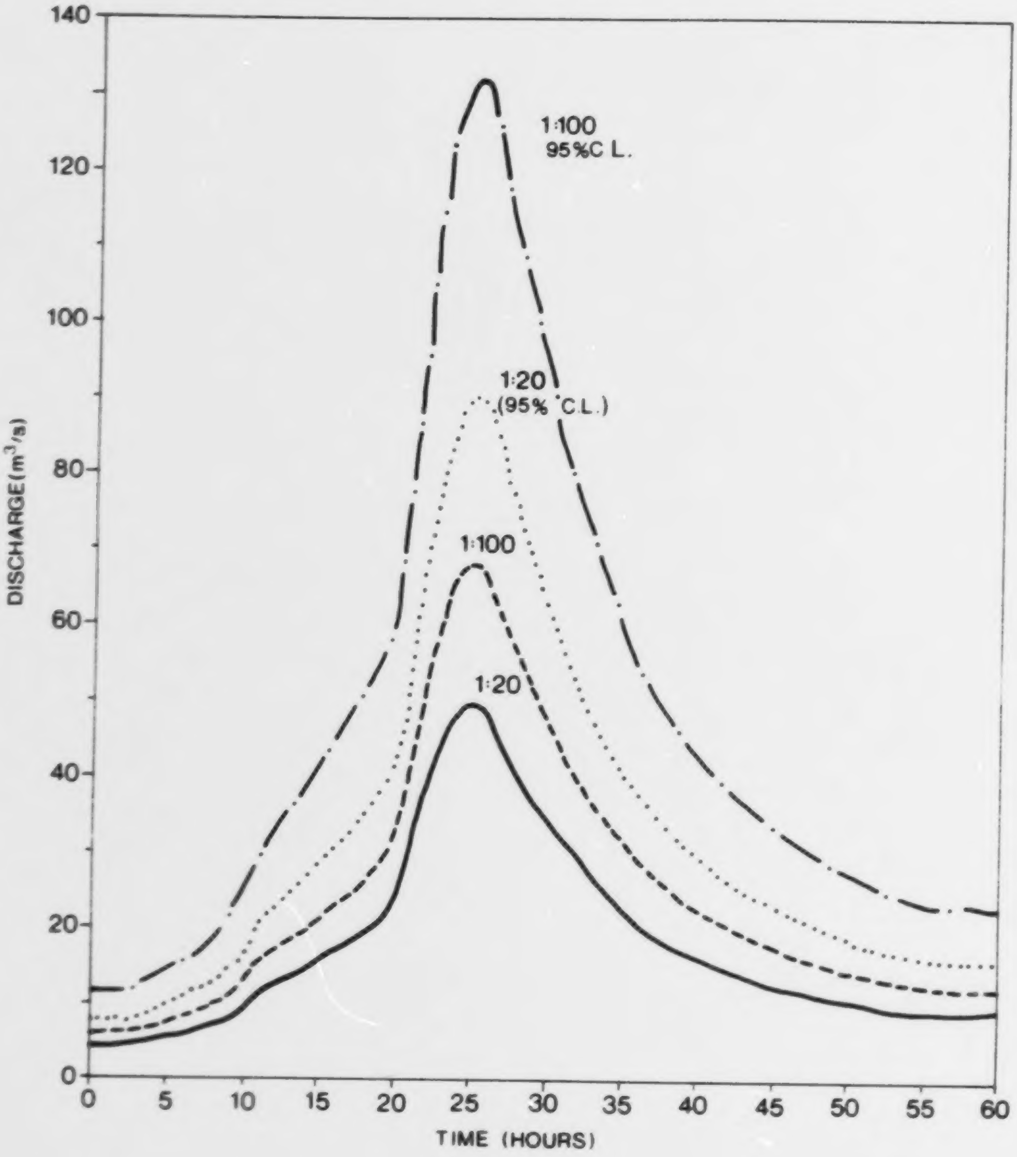
CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

TRURO HYDROTECHNICAL STUDY

DESIGN FLOOD HYDROGRAPHS FOR SOUTH BRANCH, NORTH RIVER

ABOVE CONFLUENCE WITH SOUTH BRANCH

FIGURE 6.4



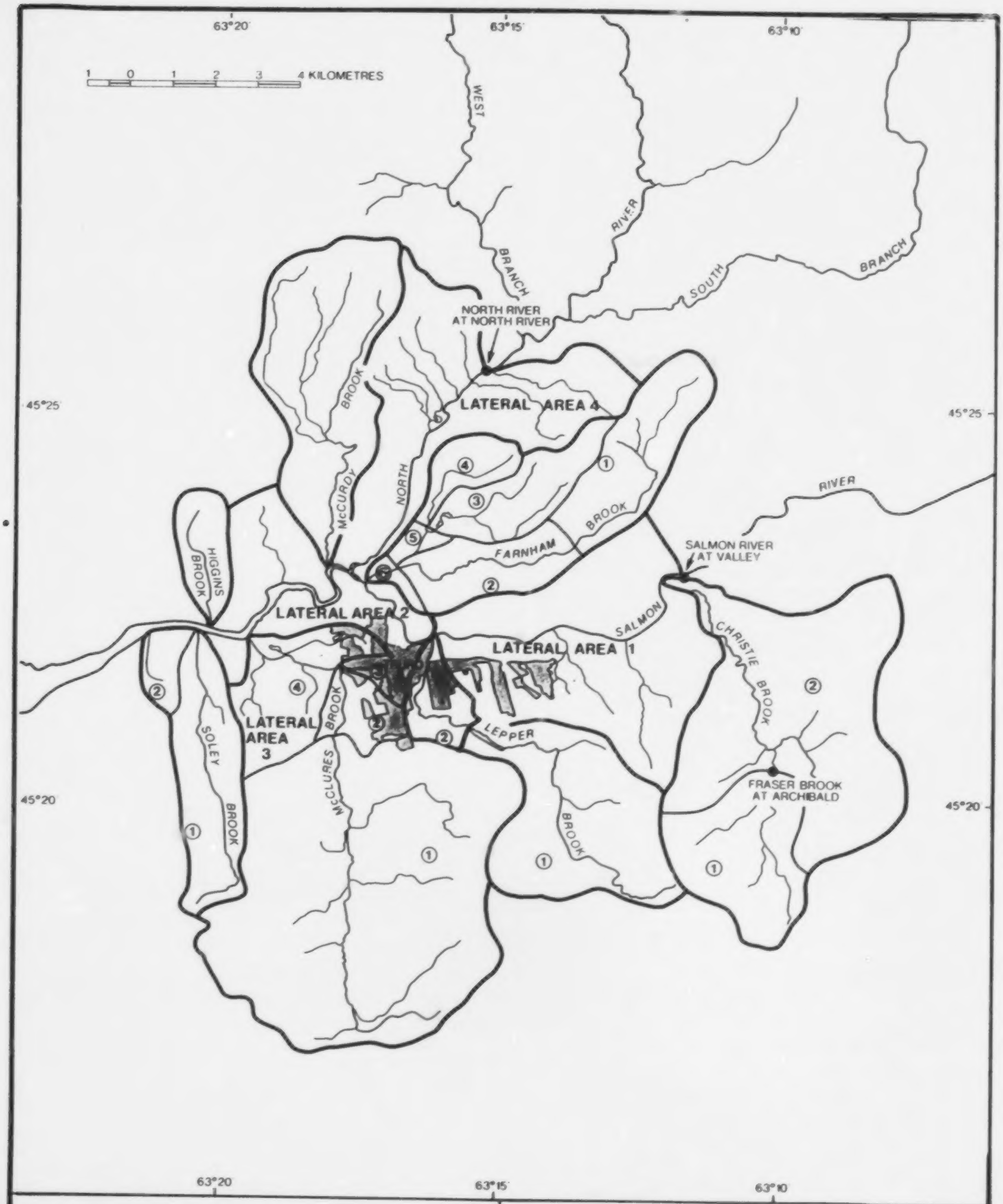
CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

TRURO HYDROTECHNICAL STUDY

DESIGN FLOOD HYDROGRAPHS FOR WEST BRANCH, NORTH RIVER

ABOVE CONFLUENCE WITH WEST BRANCH

FIGURE 6.5


63°20'
63°15'
63°10'
1 0 1 2 3 4 KILOMETRES
WEST BRANCH
NORTH RIVER
SOUTH BRANCH
NORTH RIVER AT NORTH RIVER
45°25'
LATERAL AREA 4
McCURDY BROOK
NORTH
HIGGINS BROOK
FARNHAM BROOK
RIVER
SALMON RIVER AT VALLEY
LATERAL AREA 2
SALMON
LATERAL AREA 1
CHRISTIE BROOK
LATERAL AREA 3
BROOK
LEPPER BROOK
SOLEY BROOK
McCLURES
FRASER BROOK AT ARCHIBALD
45°20'
①
②
③
④
⑤
⑥

LEGEND

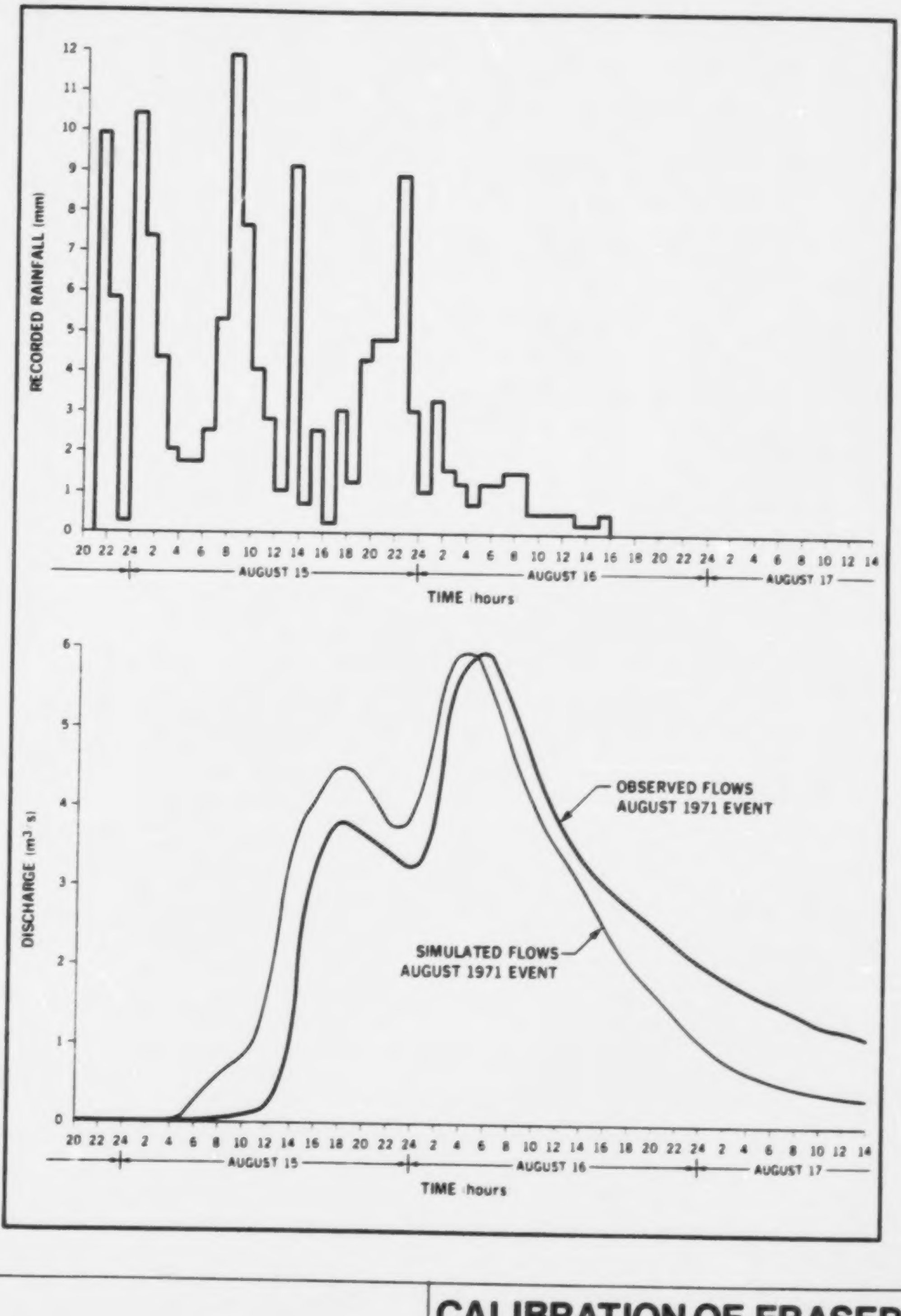
● **WATER SURVEY OF CANADA GUAGE**

② **SUBWATERSHED DESIGNATION**

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

TRIBUTARIES IN THE TRURO AREA

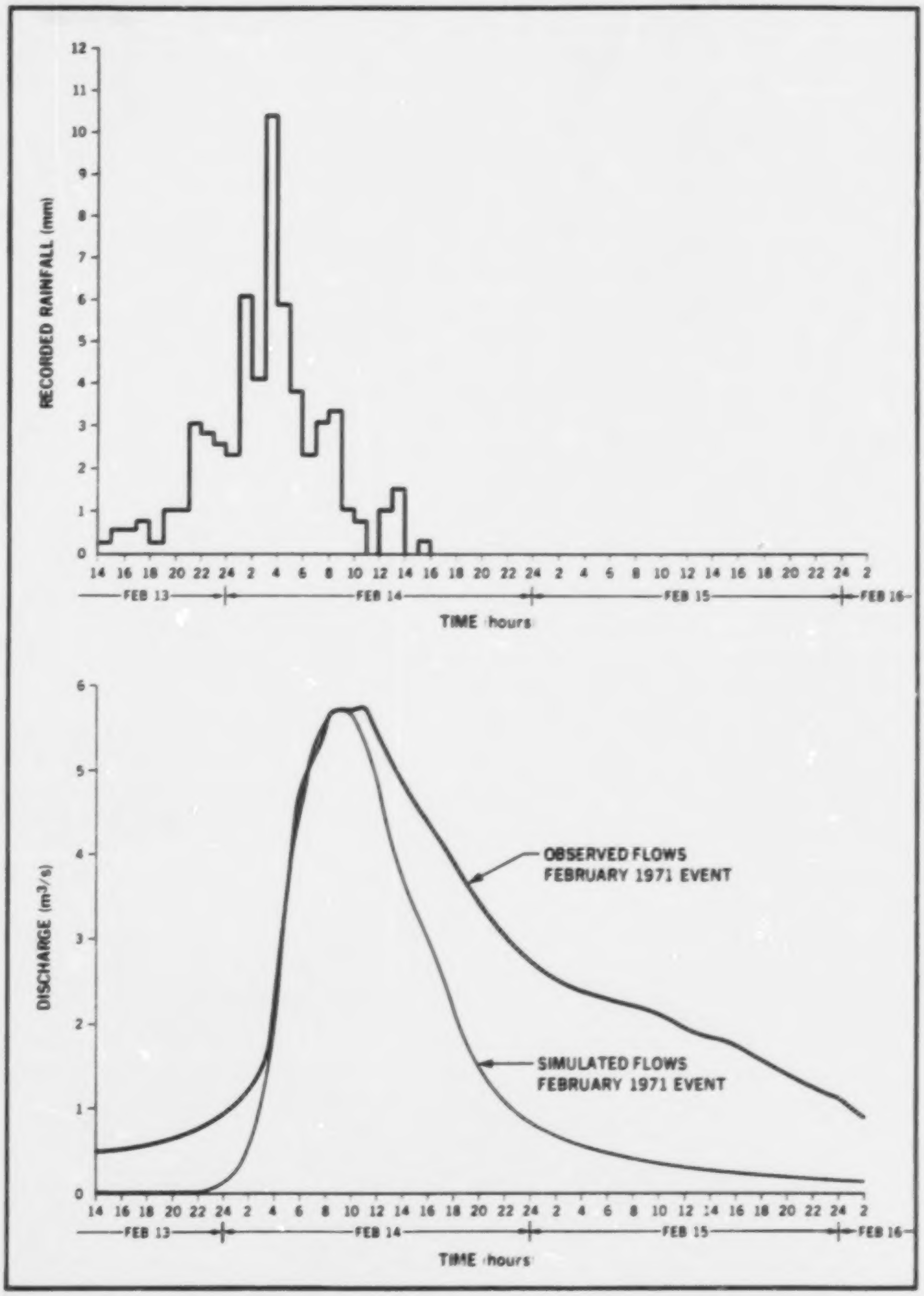
FIGURE 6.6



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

CALIBRATION OF FRASER BROOK MODEL,
AUGUST 1971 STORM EVENT

FIGURE 6.7

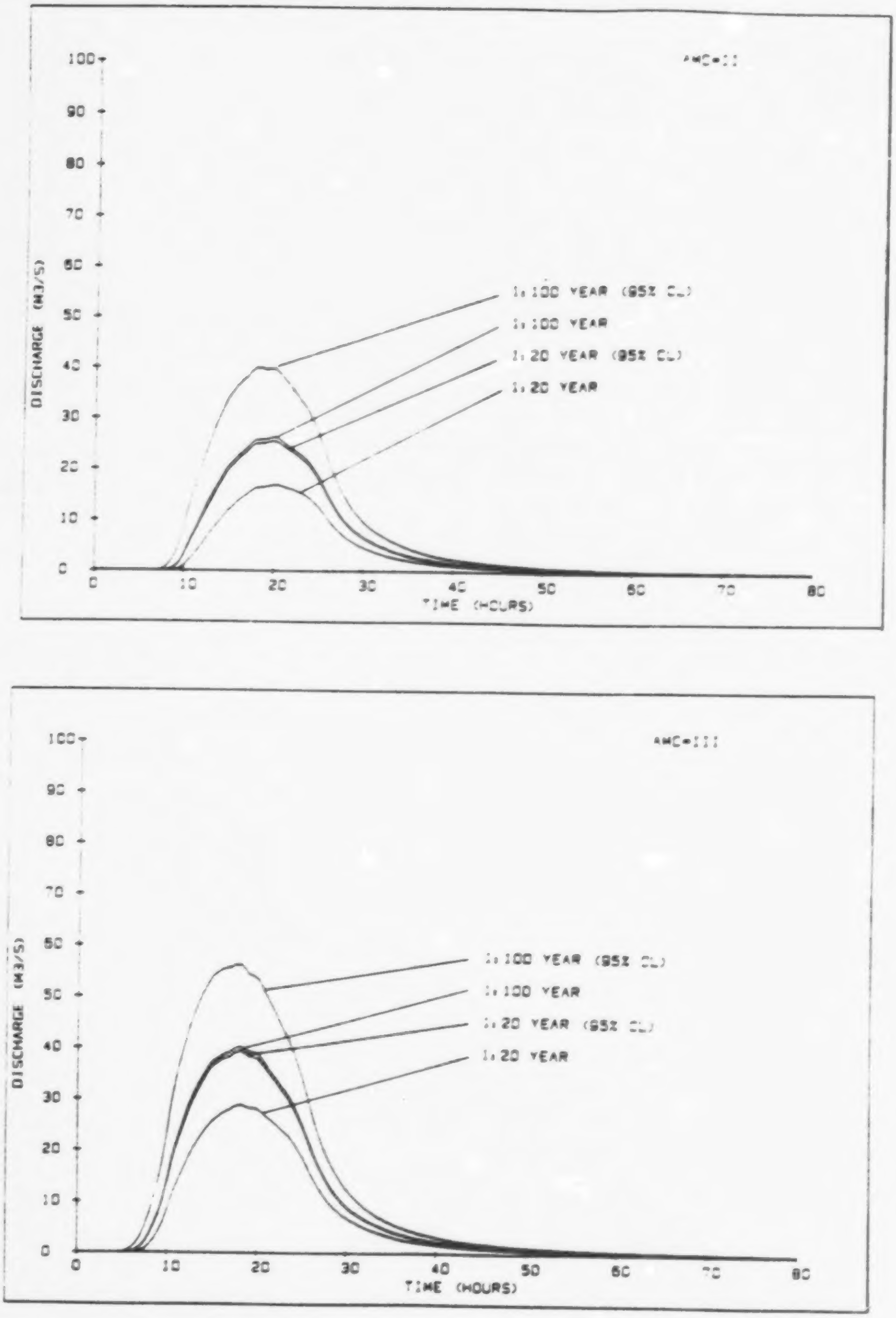


CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

VERIFICATION OF FRASER BROOK MODEL,

FEBRUARY 1971 STORM EVENT

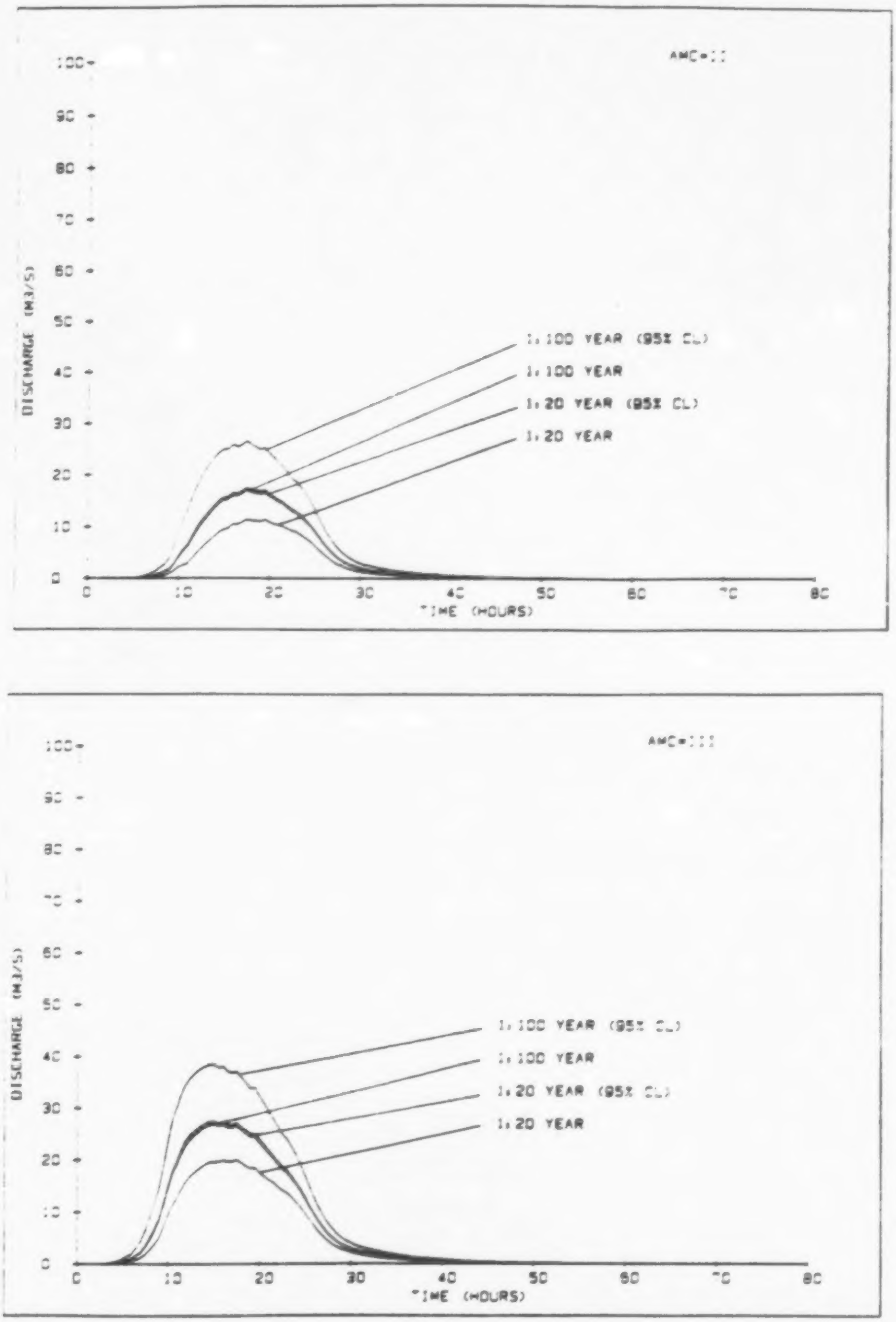
FIGURE 6.8



DESIGN FLOOD HYDROGRAPHS FOR CHRISTIE BROOK

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

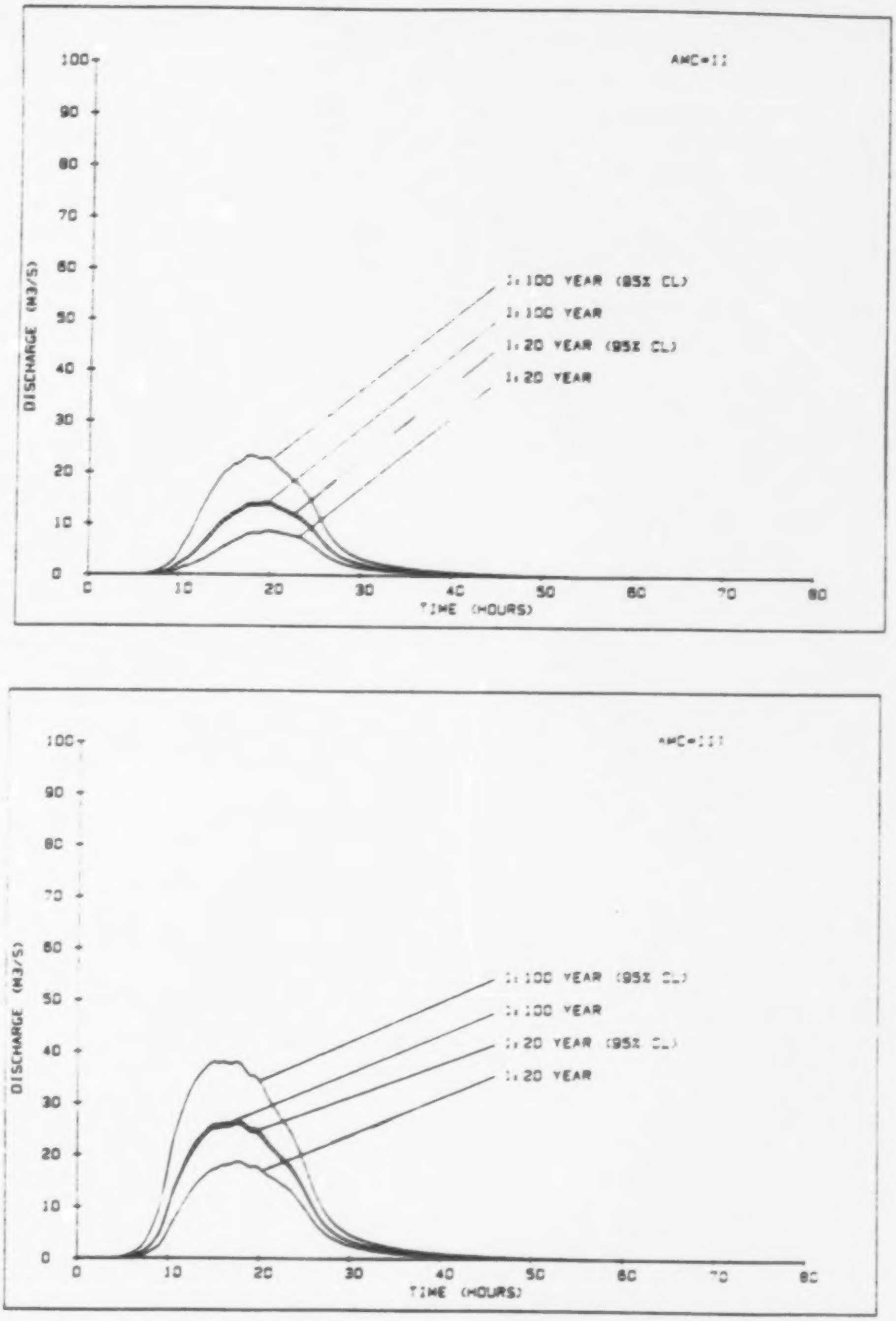
FIGURE 6.9



DESIGN FLOOD HYDROGRAPHS FOR LEPPER BROOK

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

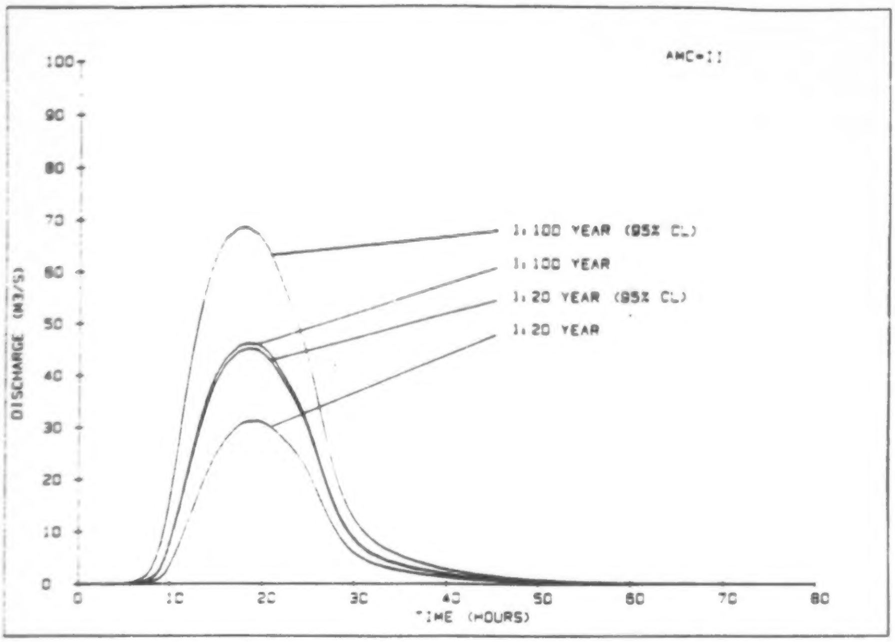
FIGURE 6.10

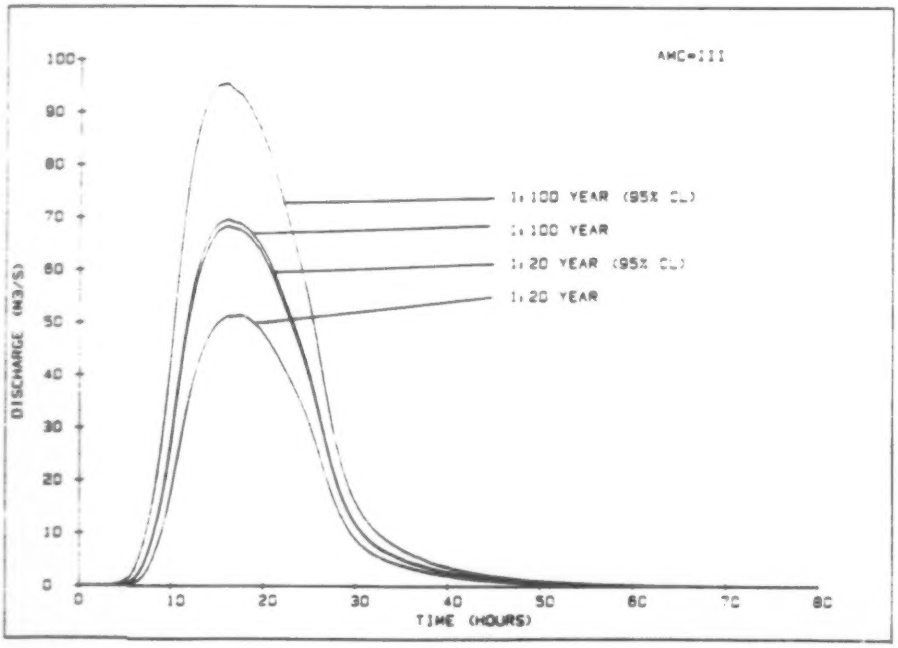


DESIGN FLOOD HYDROGRAPHS FOR FARNHAM BROOK

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 6.11

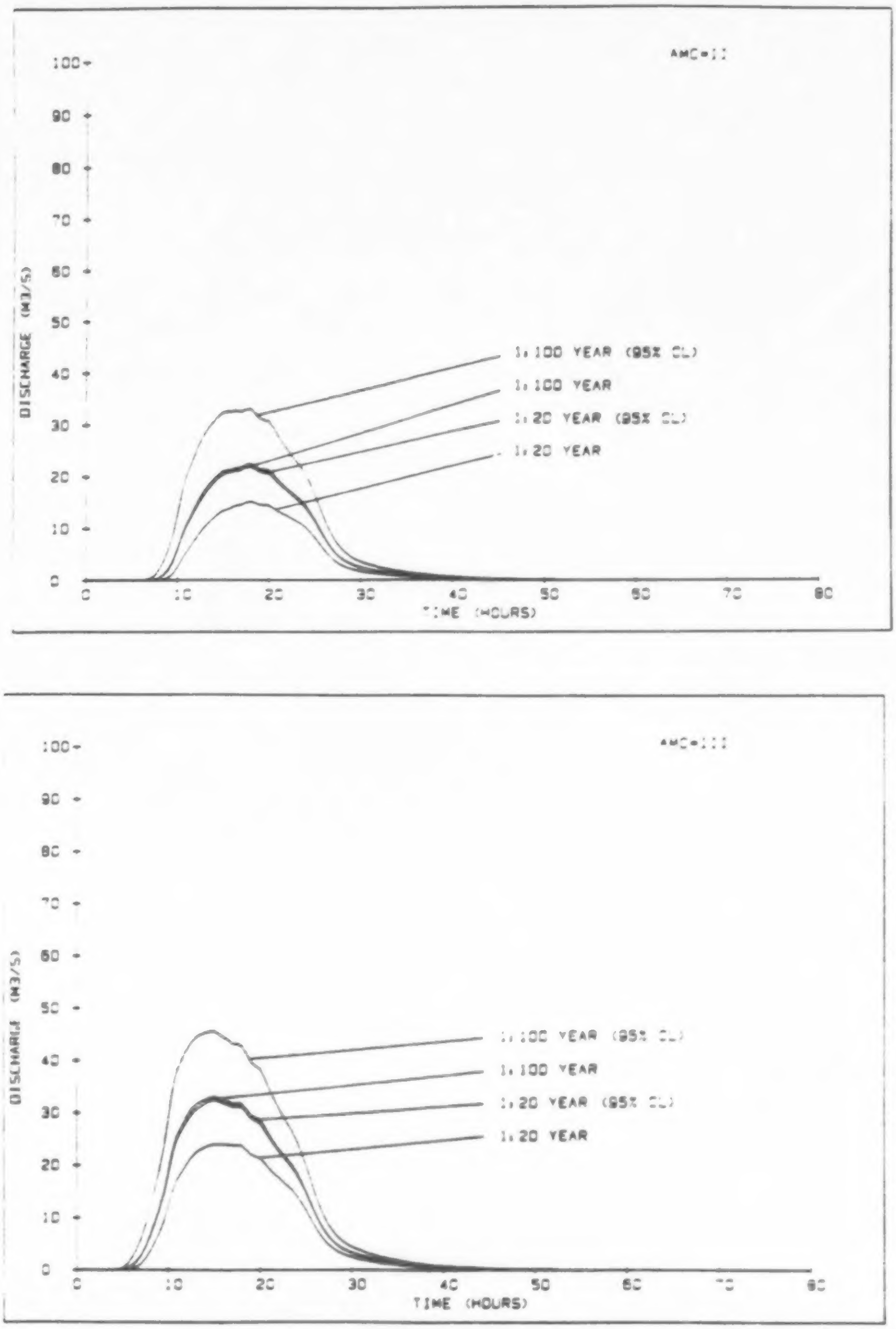




DESIGN FLOOD HYDROGRAPHS FOR McCLURES BROOK

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

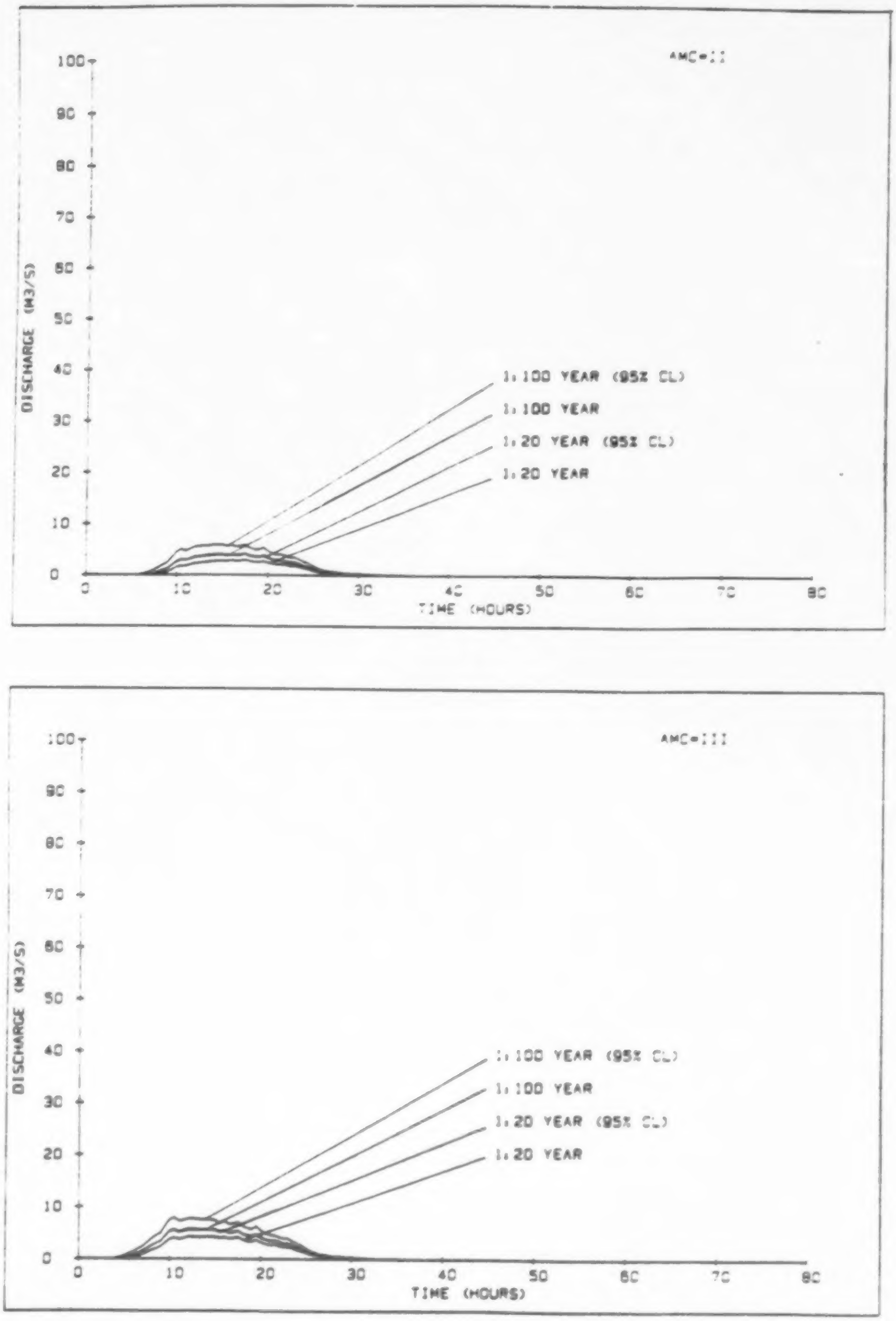
FIGURE 6.12



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

DESIGN FLOOD HYDROGRAPHS FOR McCURDY BROOK

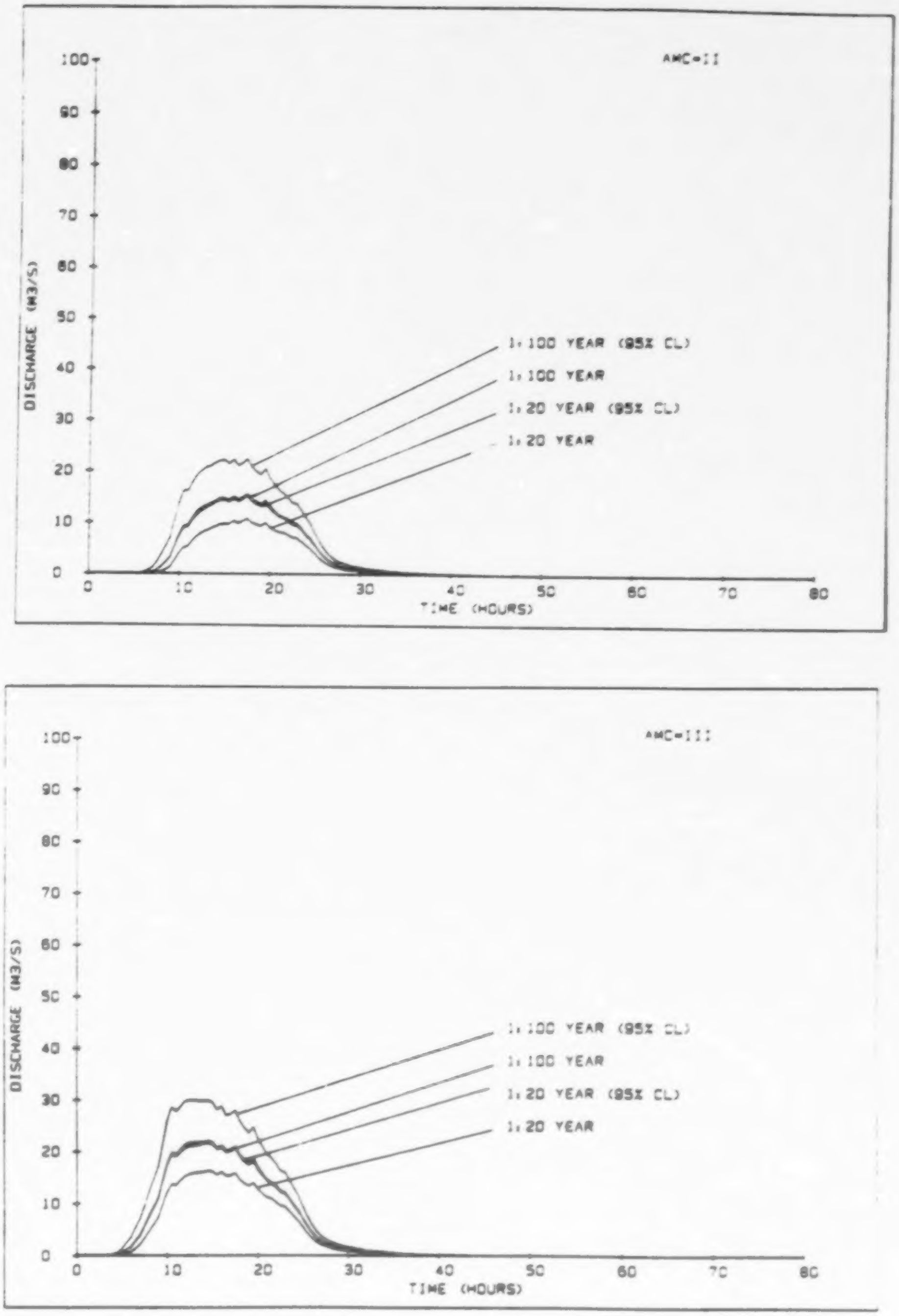
FIGURE 6.13



DESIGN FLOOD HYDROGRAPHS FOR HIGGINS BROOK

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 6.14



DESIGN FLOOD HYDROGRAPHS FOR SOLEY BROOK

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 6.15

7.0 TIDAL ANALYSES

7.1 General Description of the Bay of Fundy Tides

7.1.1 Introduction

The magnitude of tides in the Bay of Fundy are among the highest in the world. Tidal characteristics have been described in a number of previous reports (see Chapter 4.0). At Burntcoat Head, the closest location to Truro on the Minas Basin where the complete tidal cycle can be conveniently measured, the tidal range has been measured at 16.3 m during June-July 1916 (Bell-Dawson, 1917) and 16.1 m during August-September 1960 (Atlantic Tidal Power Programming Board, 1969). These time periods would not have provided the maximum range based on astronomic factors, but nearly so.

Unfortunately, insufficient data are available to adequately describe the tidal regime of the Bay of Fundy in general, and of the Minas Basin in particular. During the course of this study, it was necessary to assemble and interpret data from a variety of sources, not all of which can easily be related in time and space. However, despite the complexities of the tides and the paucity of data, tides are quite regular and can, with few exceptions, be predicted many years in advance. These exceptions include (i) atmospheric disturbances, and (ii) areas along shallow water bays and estuaries where the tide is distorted by channel topography and bottom friction. Since both of these factors are of significance in the Truro area, additional investigations were required.

7.1.2 Available Tidal Information

Systematic tidal records on the Bay of Fundy date back to 1896 when a permanent tidal gauge was established at Saint John, New Brunswick. The annual extreme water levels available for the tidal gauge at Saint John are tabulated in Table 7.1. Since this gauge provides the only continuous record of tides in the Bay of Fundy,

TABLE 7.1

Yearly Tidal Extremes for Station 0065 Saint John[2]

Elevations in Metres Referred to Chart Datum[1]

Year	High	Low	Year	High	Low
1900	8.87	-0.24	1901	8.75	-
1902	8.81	-	1903	-	-
1904	-	-	1905	-	-0.30
1906	-	-	1907	-	-0.18
1908	8.75	-	1909	-	-0.18
1910	8.69	-	1911	-	-
1912	8.63	-0.06	1913	-	-
1914	8.78	-0.24	1915	8.66	-
1916	8.81	-0.15	1917	8.81	-0.27
1918	8.90	-0.27	1919	8.84	-0.09
1920	8.66	-0.18	1921	8.53	-0.15
1922	8.69	-0.24	1923	8.84	-0.30
1924	8.66	-0.30	1925	8.53	-0.27
1926	8.60	-0.15	1927	8.66	-0.24
1928	8.66	-0.34	1929	8.53	0.00
1930	8.66	-0.15	1931	8.66	-0.24
1932	8.69	-0.40	1933	8.93	-0.30
1934	8.81	-0.15	1935	8.63	-0.18
1936	8.81	-0.18	1937	8.78	-0.15
1938	8.69	-0.34	1939	8.78	-0.24
1940	8.78	-0.24	1941	8.78	-0.34
1942	8.63	-0.30	1943	8.78	-0.37
1944	8.66	-0.43	1945	9.02	-0.09
1946	8.75	-0.03	1947	8.72	+0.03
1948	8.78	-0.15	1949	8.84	-0.03
1950	8.69	-0.12	1951	9.02	-0.06
1952	8.90	-0.15	1953	9.02	-0.21
1954	8.81	-0.15	1955	8.78	-0.12
1956	9.02	-0.24	1957	8.53	-0.18
1958	8.81	-0.06	1959	8.72	-0.12
1960	8.66	-0.15	1961	8.69	-0.03
1962	8.90	-0.24	1963	8.93	-0.34
1964	8.72	-0.12	1965	8.78	-0.18
1966	8.66	-0.21	1967	8.78	-0.08
1968	8.72	-0.09	1969	8.60	-0.09
1970	8.69	-0.03	1971	9.02	+0.03
1972	8.75	-0.03	1973	8.81	-0.02
1974	8.72	-0.14	1975	8.81	-0.18
1976	9.14	-0.09	1977	9.22	-0.18
1978	9.14	-0.09	1979	9.02	-0.15
1980	8.90	-0.24	1981	8.78	-0.16
1982	8.65	-0.12	1983	8.52	-0.12
1984	8.84	-0.20	1985	8.77	-0.11

1. Geodetic Datum is 4.19 metres above Chart Datum
2. Source: Marine Environmental Data Service, Canadian Hydrographic Service, Fisheries and Oceans Canada

it is the basis for tidal predictions issued by Fisheries and Oceans Canada. Through the course of time, temporary tidal records have been obtained at some 49 secondary ports in the Bay of Fundy. Based on relationships between each of these secondary ports and the reference port at Saint John, it is possible to predict astronomic tides at each site. The closest secondary port, and the only one in Cobequid Bay, is at Burntcoat Head. These relationships, along with tidal predictions at the reference port, are publshed annually by the Canadian Hydrographic Service, Fisheries and Oceans Canada.

The Maritime Marshlands Reclamation Agency (a predecessor to the present day Maritime Resource Management Service) has, over the years, collected tidal data at a number of locations. Peak tidal levels in the Truro area during high tide periods in 1958, 1959 and 1961 are presented in Table 7.2. Data for selected flood events in the Truro area are also available and are presented in Chapter 5.0.

The most recent tidal observations in the Cobequid Bay were carried out by the Canadian Hydrographic Service in 1976 (DeWolfe, 1976/77). A continuous record of the tides was obtained over a 90 day period at a location approximately 5 km east of Burntcoat Head. Two additional gauges were located in Minas Basin, one near Economy Point and the other near Cape Blomidon. These data were collected during a period of high tides, and consequently they are potentially of considerable value in the context of predicting high tide elevations. However, subsequent analyses of these data failed to show any improvement in the level of understanding of the tidal regime in Cobequid Bay (Godin et al, 1981).

During the ice free periods of 1971 and 1972, the Nova Scotia Department of Agriculture, with the support of the Canadian Hydrographic Service, Fisheries and Oceans Canada, established and operated a continuous water level gauge at the site of the Old Board Landing Bridge. These observations, the first of their kind in the Truro area, were invaluable to resolve the tidal characteristics of

TABLE 7.2

Miscellaneous Peak Tide levels Measured in the Salmon River Estuary[1]

Date Year	Mo.	Day	Board Landing Bridge[2] (m)	Onslow North River Aboiteau #8[2] (m)	Park Street Bridge[2] (m)	Corresponding Saint John, N.B. Predicted Tide [3,4] (m)
1958	04	04	9.61		9.74	8.87
	04	05	9.53			8.69
	04	06	9.56	9.54	9.72	8.87
	05	03	9.25			8.75
	05	05	9.25			8.75
	10	13	9.33	9.36	9.45	8.78
	10	14	9.27	9.36	9.45	8.87
	10	15	9.15	9.21	9.24	8.72
	11	11	9.45			8.81
	11	12	9.45			8.84
	11	13	9.27		9.48	8.69
1959	04	23	9.36	9.42	9.48	8.84
	04	25	9.45	9.48	9.57	8.87
	05	22	9.39	9.45	9.57	8.84
	05	24	9.30			8.84
	11	30	9.36	9.42	9.57	8.78
	12	01	9.45	9.51	9.63	8.84
	12	02	9.42	9.57	9.62	8.78
1961	03	17	8.60 (ice)			8.54
	09	22	8.90			8.17
	09	23	9.21			8.41
	09	24	9.27			8.54
	09	25	9.30			8.54

1. Adopted from Desplanque (1977)
2. Elevation in metres (Geodetic Datum)
3. Elevation in metres (Chart Datum)
4. To convert to Geodetic Datum, subtract 4.19 metres

the Salmon River Estuary during the course of this study. The shallow water effects of Cobequid Bay and the Salmon River Estuary were very evident in these data. Referring to Figure 7.3, the initial response is one of a rapid rise in water level followed by a more gradual decline in relation to the sinusoidal signature of oceanic tides. These data have been analysed by a number of investigators, including Inland Waters Directorate, 1978; Desplanque, 1977; and Godin et al, 1981. A digital and analog record of these data have been archived by the Marine Environmental Data Services Branch of Fisheries and Oceans Canada.

Previous studies on the tidal regime were undertaken by a number of investigators and are discussed in Chapter 4.0.

7.1.3 Astronomical Factors Affecting Tides (1)

Tides are caused primarily by a complex interplay of the gravitational forces of the sun, the moon and the earth. The magnitude of the tide is affected by the relative position of all three bodies in relation to each other. As the earth rotates, the gravitational forces of the sun pull water on either side of the earth causing two tidal cycles per day, each with a period of 12 hours. Since the earth moves in an elliptical orbit around the sun, the forces vary depending on the time of year; being stronger in January when the earth is closest to the sun, and weaker in July when the earth is farthest from the sun.

As the earth rotates on its axis, the moon is moving counterclockwise around the earth. Each day it moves a little further east, so it takes 24 hours and 50 minutes to reach the previous day's position. Since the moon also pulls water on either side of the earth, there are two tidal cycles per day, each with a period of 12 hours and 25 minutes. The gravitational attraction of the moon is

(1) Adapted from Roland, 1982, and Godin et al, 1981

stronger than the sun, and therefore, the lunar tide dominates. The path of the moon around the earth is also elliptical. The forces of the moon on the tide reaches a maximum every 27.55 days (Perigean Tide).

When the earth, moon and sun are closely aligned, the gravitation forces combine to produce "spring tides". Conversely when the moon is at first and third quarter, the lunar and solar tides are opposing, creating low or "neap tides". The resulting pattern is illustrated in Figure 7.1. When the moon is in perigee with the earth during full and new moons (nearest point on its orbit), the tidal amplitudes are further accentuated. This occurs every 6.75 months (spring Perigean and Apogean Tides).

The factors described above cause twice-daily or semi-diurnal tides of equal magnitude. However, inequalities in successive tides are evident in tidal records and are explained by the declination of the sun, which varies to the degree that the orbits of the moon and the earth are at an angle with the plane of the earth's equator. This gives rise to daily or diurnal variations in the tide. The maximum diurnal tide in the Bay of Fundy will tend to occur when the moon is at its largest angle north of the equator and in line with the sun at new or full moon (Declinational Spring Perigean Tides). This alignment occurs regularly once every 18.6 years, last occurring in 1976, and next in 1995.

The tides in the Bay of Fundy can be approximated by calculating the resultant of a number of separate tidal constituents, the most significant of which are described above, each having a different range, frequency and pattern. For example, tides are predicted at Saint John Harbour, the primary reference port for the Bay. Tidal predictions at Saint John are based on the resultant of some 18 different tidal constituents and are accurate to within ±0.30 m during winter months, and ±0.15 m during summer months. Secondary predictions are also made at a number of other ports along the Bay, based on empirical relationships developed over the years (e.g. Burntcoat Head is one of the secondary ports).

7.1.4 Tidal Propagation

The Bay of Fundy is a body of water open to tidal influences of the Gulf of Maine. Water in any such basin has a natural period of oscillation as tides cause it to flow back and forth. Once set in motion, it moves somewhat like a pendulum and only a small push is needed to overcome bottom friction and maintain its amplitude. The water in the head of the Bay reverses flow direction, reaching maximum velocity at mean tide in the Bay and low tide outside. The momentum from this flow carries the water out of the Bay for an additional three hours, at which time the rising Gulf of Maine tide gives another push to maintain the oscillation. The natural period of oscillation in the Bay appears to be in harmony with the lunar semi-diurnal constitutents of the tide. Therefore, lunar tidal constituents are the dominant forces behind Bay of Fundy tides in view of the fact that those constituents found in the oceanic tides of the North Atlantic are further amplified by the physical properties of the Bay (Roland, 1982).

Others have suggested that the convergence of the Bay from its mouth to its head, both laterally and vertically, is also important in creating large tides (Atlantic Tidal Power Programming Board, 1969). As the tide moves up Cobequid Bay, the oceanic tide gradually becomes distorted by topographic influences and by friction, and ultimately it becomes a progressive wave. As the tide initiates its movement up a tidal estuary, such as the Salmon River, a breaking wall of water, called a tidal bore, is formed. The height of the bore varies from a few millimeters to as high as 0.9 metres (Roland, 1982).

The progagation of the tides from Saint John, New Brunswick, to Burntcoat Head and ultimately to the site of the Old Board Landing

Bridge in Truro is graphically depicted on Figure 7.2. The tide at Saint John is a sinusoidal curve as is that at Burntcoat Head. The tide is amplified considerably from Saint John to Burntcoat Head and is further amplified at Truro. However, the effect of bottom topography and friction on tides is evident at Truro.

7.1.5 Atmospheric Disturbances

Atmospheric disturbances, such as strong or prolonged winds, abrupt changes in barometric pressure, or prolonged periods of very high or very low pressures, introduce fluctuations (storm surges) in sea level. The impact of storm surges on peak levels is very much dependent on when they occur in relation to astronomic tides. The tides of the Bay of Fundy are very large, and the tidal currents strong and not easily disturbed by air movement (Bell-Dawson, 1917). Currents through the Minas Channel run at a rate of about 4 m/sec, while in Cobequid Bay, currents are about half that amount (Desplanque, 1977). If a significant meteorologic disturbance occurs during low or falling tides, then the chances are that it will cause a phase change in the tide, but little or no increase in peak levels. On the other hand, if the meteorological disturbance is of sufficient duration, or occurs when the tide is rising, the resultant effect can have significant consequences, particularly during spring tides.

Generally, the surface of the ocean responds to atmospheric pressure changes like an inverted barometer, and can be approximated by the following equation:

$$SSp = 1.02\,\Delta P \qquad (7.1)$$

where SSp = Storm surge due to pressure change in centimetres
ΔP = Deviation of pressure from normal in millibars.

As a pressure system moves across a body of water, it generates forces through the associated wind and pressure fields which in turn modify the tides. As indicated in Section 7.1.4, a body of water has its own natural period of free oscillation determined by its depth, shape and size. It may be shown that if the speed of the motion of the pressure system approaches the rate of travel of a free oscillation, resonance occurs and the effect of the pressure system on sea level will be amplified. The rate of travel of a free oscillation (in knots) can be estimated as $8.23 \times \sqrt{\text{depth in fathoms}}$ (Russel and MacMillan, 1952). For the Bay of Fundy, the average depth is approximately 40 fathoms (73 m). Thus the resonance effect should be strongest for a pressure system moving at about 52 knots (95 km/hr) along the axis of the Bay. During the famous "Groundhog Day" Storm of February 2, 1976, the low did move at very nearly this speed. Combined with the strong winds up the axis of the Bay and low surface pressures, the effect of this storm was to give very high storm surges at both Saint John and Yarmouth (Galbraith, 1979).

The degree to which the tides are set up by wind is directly proportional to wind speed/direction and inversely proportional to water depth. The fact that westerly winds dominate in the Bay of Fundy region and that Cobequid Bay is shallow, suggests that storm surges associated with wind will be more significant than those associated with barometric pressure changes. Storm surges in excess of two metres have been recorded at Saint John Harbour during the period 1947-1975 (Galbraith, 1979) and at other locations along the Bay of Fundy during the "Groundhog Day" storm in 1976 (Desplanque, 1977).

The generation of waves, affected by wind speed/direction and fetch length, is of concern in most ocean environments. In fact, the Atlantic Tidal Power Programming Board (1969) concluded that waves in excess of four metres are possible in Minas Basin. However, as waves enter Cobequid Bay, their height is greatly diminished as they break in shallow water. It is likely that by the time the waves arrive at the Salmon River Estuary, they are insignificant in relation to the combination of other meteorolgical disturbances and high astronomic tides. Desplanque (1977) concluded that "It appears that waves cannot be considered as causing a problem in the same order of magnitude as the combination of high water of large tides, low barometric pressure and wind set-up".

7.1.6 Coastal Submergence Along the Bay of Fundy

The inundation of land by the sea along the Bay of Fundy shoreline has been attributed to a number of factors, including a world-wide increase in sea level, land subsidence caused by the additional water loading resulting from increases in sea level, increases in tidal amplitude, and various geological phenomena including geosynclinal downwarping and relaxation of a former ice-marginal bulge, caused by glaciation, located on the Scotian Shelf (Grant, 1970).

There is considerable evidence of coastal submergence along the Bay of Fundy shoreline and in bottom sediments therein. Using tidal data recorded in Saint John Harbour since the turn of the century, Grant concluded that the net rate of coastal submergence was 46 cm/century. This is consistent with evidence gathered by Grant near Fort Beausejour, where a corduroy road built some 300 years ago is now submerged by approximately 1 m (35 cm/century) of salt marsh mud. Tree stumps, dated as being approximately 4000 years old using carbon 14 techniques, were found approximately 11.5 m (30 cm/century) below intertidal mud.

Vanicek (1976) undertook an analysis of vertical crustal movement in Maritime Canada using tidal data collected at eight sites, including Saint John, New Brunswick, and over 200 segments of first order levelling data obtained by the Geodetic Survey of Canada over the period 1909 to 1973. A comparison of the two sets of first order levelling data between GSC monuments obtained up to 65 years apart, served not only to substantiate conclusions drawn using tidal data but also provided a basis for extrapolting data to specific sites along the Bay of Fundy, including Truro. Based on Vanicek's analysis, the rate of coastal submergence was calculated at approximately 40 cm/century at Saint John, New Brunswick and approximately 50 cm/century at Truro, Nova Scotia.

In conclusion, coastal submergence along the Bay of Fundy is a complex phenomenon caused by a number of factors. Grant concluded that the Bay of Fundy coastline has been submerged by up to 15 m over the last 4000 years (approximately 40 cm/century). Vanicek supports this conclusion, and estimated that the rate of coastal submergence in the Truro area has been occurring at a rate of approximately 50 cm every 100 years.

7.2 Design Tides

7.2.1 General

The objective of this tidal analysis is to provide estimates of 1:20 and 1:100 year recurrence interval tidal hydrographs at the downstream boundary of the hydrodynamic model. Since it would be difficult, if not impossible, to establish a unique stage-discharge relationship in the tidal estuary, and since a significant portion of the floodplain is affected by tides, it was necessary to establish the downstream boundary of the hydrodynamic model at a location where (i) the tidal hydrograph can be predicted with some confidence; and (ii) the tidal hydrograph is unaffected by freshwater flow.

In the absence of direct measurements, it was necessary to predict both the magnitude and characteristics of the tidal hydrographs associated with each of the design events at the downstream boundary. In order to accomplish this, it was necessary to; (i) establish methods to predict astronomical and meteorological tides; and (ii) establish appropriate combinations of astronomical and meteorological tides to reflect 1:20 and 1:100 year recurrence interval design conditions.

The magnitude of the design tides was determined using the following approach:

1. assigned frequencies of the astronomic component of the tide at Saint John were determined as reported by Godin et al
2. the appropriate regression equation relating tides at Truro to those at Saint John was selected. The astronomical tide determined in 1) above was then transferred to the site of the Old Board Landing Bridge in Truro
3. a statistical analysis of the effects of meteorologic disturbances on the tidal regime at the site of the Old Board Landing Bridge was carried out
4. a joint probability analysis of astronomical and meteorological tides was carried out resulting in estimates of 1:20 and 1:100 year recurrence interval tide peaks at the site of the Old Board Landing Bridge
5. the design tides at the site of the Old Board Landing Bridge were adjusted further for the effects of coastal submergence
6. the magnitudes of the design tides at the site of the Old Board Landing Bridge were adjusted to the downstream boundary at McNutt Brook
7. design tidal hydrographs were then determined for the McNutt Brook location by utilizing the classical sinsuoidal tidal variations as recorded at Burntcoat Head.

The selection of the downstream boundary location is discussed in the following section. Subsequent sections describe the determination of various factors affecting the magnitude of the design tidal levels, including astronomical tidal characteristics, effects of meteorological disturbances, the joint probability of the previous two factors, and the influence of coastal submergence. Finally, design tidal elevations at McNutt Brook are given in Section 7.2.8 and the methodology and results for determining the design tidal hydrographs are discussed in Section 7.2.9.

7.2.2 Selection of the Downstream Boundary Location

Clearly, there would be many advantages associated with selecting the downstream boundary at the site of the former tidal gauge located at the Old Board Landing Bridge since the historical measurements could be used to synthesize design tides at that location. However, freshwater flow was found to have a significant influence on the shape of the tidal hydrograph at this location. This is demonstrated with reference to Figure 7.4, which indicates that near the time of occurrence of the peak freshwater flows, the normal shape of the tidal hydrograph (which is truncated due to the high elevation of the bottom of the estuary at this location) is significantly distorted.

The impact on tides of two peak flow events (May 5, 1972 and August 16, 1971) is demonstrated on Figure 7.4. During Hurricane Beth, which occurred August 14-17, 1971, peak flood flows marginally exceeded the 1:20 year recurrence interval flood flow estimate on the Salmon River. During the event of May 4-6, 1972, peak flows exceeded those previously recorded, and approached a 1:50 year recurrence interval. However, for the latter event, the recorded hydrograph shows somewhat less distortion in shape near the occurrence of the peak flow. This is attributed to the timing and volume of runoff during Hurricane Beth compared to that of the May 1982 event.

The predicted tides at Truro, based on a regression equation relating Truro elevations to recorded observations at Saint John, are also plotted in Figure 7.4 for each event. The difference between recorded stage at Truro and that predicted at Truro, using recorded data at Saint John, is tabulated on Table 7.3. During the August 1971 event, the prediction equation generally underestimated stage at Truro, while just the opposite occurred during the May 1972 event. Based on these data, it is difficult to conclude that freshwater flow materially affected peak levels at Truro. However, it is definite that the shape of the tidal hydrograph is affected by freshwater flow, and therefore, it was concluded that a site further downstream must be selected as a boundary for the hydrodynamic model.

It is likely that the tidal regime at Burntcoat Head (or Bass River) would not be materially affected by freshwater runoff in view of the sheer size of the Bay. However, there was some reluctance to choose this location for two reasons: (i) the cost of modelling the tidal regime between Burntcoat Head and the Salmon River Estuary; and (ii) the doubtful accuracy that would be achieved in view of the limited hydrometric and bathymetric data, particularly with regard to the latter, when the unstable geomorphological regime of the Bay of Fundy is considered.

In the opinion of the Flood Risk Mapping Technical Committee, the tidal hydrograph would not be materially affected by freshwater flow at McNutt Brook in that the estuary substantially fans out at this location. This was supported by Mr. C. D. Desplanque of the Maritime Resource Management Service (personal communication) who has significant familiarity with the study area. Unfortunately, there are no data to substantiate this conclusion other than some limited observations taken during the event of April 16, 1980 by Dillon-Horner at Soley Brook (upstream of McNutt Brook). These measurements suggest that the shape of the tidal hydrograph is closer to the shape of the tidal hydrograph recorded at Bass River than that recorded at the Old Board Landing Bridge (see Figure 7.5).

A number of investigators, including Dillon-Horner and Desplanque, expressed concern that tidal elevations above the dykes in the Salmon River Estuary should not be predicted based on correlation of data at Saint John and Truro (Old Board Landing Bridge). This concern was raised since the dykes had not been overtopped during the period of coincident record and since the overtopping of the dykes would have a significant impact on the relationship derived. It was concluded that the impact of dykes along the Bay of Fundy

TABLE 7.3

Comparison of Predicted and Actual Peak Elevations

Old Board Landing Bridge - Truro

Date	Recorded Elevation Saint John (Chart Datum) (m)	Predicted* Elevation Truro (Geodetic Datum) (m)	Recorded Elevation Truro (Geodetic Datum) (m)	Recorded-Predicted Elevation Truro (m)
1971 Aug. 13 (p.m.)	7.68	7.77	7.92	0.15
14 (a.m.)	7.04	6.74	7.10	0.36
(p.m.)	7.44	7.38	7.44	0.06
15 (a.m.)	6.92	6.52	6.74	0.22
(p.m.)	7.28	7.13	7.01	-0.12
16 (a.m.)	6.83	6.37	6.83	0.46
(p.m.)	7.28	7.13	7.35	0.22
17 (a.m.)	6.94	6.58	7.07	0.49
(p.m.)	7.53	7.53	7.59	0.06
18 (a.m.)	7.10	6.83	7.19	0.36
(p.m.)	7.65	7.71	7.83	0.12
1972 May 3 (a.m.)	7.38	7.25	6.89	-0.36
(p.m.)	7.01	6.68	6.49	-0.19
4 (a.m.)	7.44	7.38	6.89	-0.49
(p.m.)	7.04	6.74	6.61	-0.13
5 (a.m.)	7.36	7.22	6.89	-0.33
(p.m.)	6.95	6.58	6.58	0.00
6 (a.m.)	7.22	7.01	6.71	-0.30
(p.m.)	7.01	6.68	6.64	-0.04
7 (a.m.)	7.36	7.22	7.10	-0.12
(p.m.)	7.16	6.92	6.71	-0.21

* Using Equation presented in Section 4.13 (Figure 4.3)
Other equations would produce similar conclusions.

below McNutt Brook, however, would be insignificant in view of the sheer volume of water transmitted by the tide in relation to the marginal amount of water which would overtop the dykes at extreme tides. However, the impact of the dykes on water levels upstream of McNutt Brook was accounted for by the hydrodynamic model.

7.2.3 Astronomical Tide - Saint John, New Brunswick

As indicated in Section 7.1.3, the astronomical tidal regime at Saint John, New Brunswick can be predicted many years in advance to within 0.3 m during winter months, and 0.15 m during summer months. Based on an analysis of tides over the period 1981-2080, Godin et al concluded that the maximum tide at Saint John would be 8.87 m, corresponding to a recurrence interval of 1:18.6 years as reported in Section 4.15. The analysis on astronomical tidal predictions by Godin et al was accepted by the Flood Risk Mapping Technical Committee since it is one of the most complete, recent and authoritative investigations. The relevant conclusions with respect to astronomical tides are duplicated in Table 7.4.

7.2.4 Astronomical Tides - Truro, Nova Scotia

As discussed in Section 4.0, several investigators have developed regression procedures for predicting tidal elevations at Truro. In all cases, the tidal elevations recorded at Truro during 1971 and 1972 were related to either recorded or predicted tide at Saint John, New Brunswick. A comparison of the various equations is shown graphically on Figure 7.6. For prediction of high tides, the relationships proposed by Desplanque and Godin et al produce similar estimates, while that proposed by the Inland Waters Directorate would produce estimates about 0.2 m higher. All three relationships fall well within the range of recorded data. There are little or no data available to independently validate these relationships. However, miscellaneous data collected during the high tide period, 1958-61, referenced in Table 7.2, and during the periods April 16-17, 1980 and April 27, 1982, referenced in Chapter

TABLE 7.4

ASTRONOMICAL TIDES - SAINT JOHN, NEW BRUNSWICK

Elevation in Metres (Chart Datum)	Recurrence Interval	
	Months	Years
7.95	0.5	0.0417
8.11	1.0	0.0833
8.63	13	1.0833
8.87	223	18.6

TABLE 7.5

ASTRONOMICAL TIDES - TRURO, NOVA SCOTIA

Elevation in Metres (Geodetic Datum)	Recurrence Interval	
	Months	Years
8.05	0.5	0.0417
8.29	1.0	0.0833
9.08	13	1.0833
9.48	223	18.6

5.0, provide a measure of independent validation. These data are selectively plotted on Figure 7.6. These data generally fall within the range of the 1971-72 data base with a tendency towards the Desplanque and Godin et al relationship. Referring to Section 4.14, Dillon-Horner (1980) found that a high tide of 8.75 m (Chart Datum) or 4.55 m (Geodetic Datum) at Saint John would correspond to a tide of 9.45 m (Geodetic) at Truro. This agrees very closely with either the Desplanque or Godin et al relationships.

In view of the rigorous analyses carried out by Fisheries and Oceans Canada in comparison to those of the Inland Waters Directorate and Desplanque, the relationship, as presented by Godin et al, has been adopted. However, any one of the alternatives produce credible results. Furthermore, since the relationship developed by Fisheries and Oceans Canada relating predicted tidal elevation at Saint John to observed elevations at Truro produced a slightly better correlation coefficient (0.950) than did the relationship using observed tidal elevation at Saint John (0.946), and since predicted tides at Saint John are used in this study, the following relationship was adopted:

$$T_{MEDS} = 1.550Y_p - 14.024 \qquad (7.2)$$

where: T_{MEDS} = Stage in feet (Geodetic Datum) at Truro
(Divide by 3.28 to convert to metres)
Y_p = Predicted stage in feet (Chart Datum) at Saint John

It should be noted, however, that either of the Fisheries and Oceans Canada relationships could have been selected. In fact, they produce comparable results at high tidal elevations.

Based on the relationship adopted above, and the data on astronomical tides at Saint John as documented in Table 7.4, the predicted astronomical tides at Truro (Old Board Landing Bridge) are shown on Table 7.5.

The accuracy of the prediction equation has been determined to be ±0.3 m (see Section 7.1.3). It is noted that the extreme astronomical tidal prediction at the site of the Old Board Landing Bridge is below the elevation of the top of the dykes in the area (9.9 m), and is, therefore, valid from the viewpoint of the absence of the effects of dyke overtopping.

7.2.5 Effect of Meteorological Disturbances on Tides

There is a lack of information and analysis on the effect of meteorological disturbances on tides for the Bay of Fundy, and for Cobequid Bay in particular. The most comprehensive theoretical analysis was carried out by Dillon-Horner, culminating in Figure 4.5 (see Section 4.14). Godin et al carried out an analysis of "residuals" based on data recorded at the site of the Old Board Landing Bridge during ice free periods in 1971 and 1972, culminating in a series of probability plots, an example of which is reproduced herein as Figure 4.7 (see Section 4.15). The residual was defined as the difference between recorded elevation and that predicted based on a relationship between Saint John and Truro. It was assumed that the residual was due to meteorological disturbances. Selected points from each of these relationships are plotted on Figure 7.7. and a composite curve was drawn based on these data. Despite the limitations of the data base, in particular that of Godin et al and with reference to Figure 7.7, it is evident that agreement exists between these very different approaches.

The results are also consistent with Desplanque's analyses in which he concludes that a storm surge of 2.1 m at Truro is possible, but would be rare (see Section 4.10). It is not appropriate to compare estimates generated by others, such as by the Atlantic Tidal Power Programming Board, 1969, or measured by others such as the Atmospheric Environment Service and Bell-Dawson, in that there are significant differences between basic parameters such as fetch length and water depth. However, it is noted that previous estimates and records are consistent with Figure 7.7 when these basic parameters are considered. It is, therefore, concluded that the statistics of storm surges at Truro can be approximated by Figure 7.7.

7.2.6 Joint Probability Analysis of Astronomical and Meteorological Tides

Historical flood producing tides on the Bay of Fundy are usually caused by a meteorological disturbance. (Most land of any value is protected by dykes which are designed to contain astronomical tides, usually with some freeboard to spare). In order to estimate the magnitude of 1:20 year and 1:100 year recurrence interval tides, a joint probability analysis of astronomical and meteorological tides was carried out. In order to conduct such an analysis, the following two criteria must be met:

1. astronomical tides must be independent of meteorological disturbances
2. the meteorological disturbances must be of sufficient duration to affect the peak of the astronomic tide.

While research is limited, no evidence has been presented that would suggest a link between astronomical and meteorological tides (Criteria 1). There is no queston that there are meteorological disturbances that would be of such short duration that high tides would not be affected in terms of maximum elevation, although a phase shift would more than likely occur. However, for storms significant enough to produce 1:20 and 1:100 year recurrence interval tides, it seems reasonable to assume that they would be of sufficient duration to meet Criteria 2.

For each return period identified in Table 7.5, several estimates of storm surge, with varying return periods, were selected and combined in accordance with the following procedure:

$$\text{Probability (AT + SS)} = \text{Probability (AT)} \times \text{Probability (SS)} \quad (7.3)$$

where AT = Astronomical Tide at Truro in metres
SS = Storm Surge at Truro in metres

Various combinations of astronomical tide and storm surge can be computed, resulting in the same recurrence interval. A number of combinations were examined using the joint probability relationship given in Equation 7.3. Maximum values were identified and the resulting combined probability curve for tidal magnitude for the Truro area is shown graphically on Figure 7.8. Tidal estimates corresponding to 1:20 year and 1:100 year return periods for the Truro area were extracted from Figure 7.8 and summarized in Table 7.6.

7.2.7 Coastal Submergence

The elevation of tides in the Truro area in relation to the elevation of the land surface is rising at an estimated rate of 0.5 m/100 years, as discussed in Section 7.1.6. Since flood risk maps are to be used to guide development in the future, it seemed prudent to account for this phenomenon. While the length of the planning horizon could be debated, it has been assumed to be 100 years for the purposes of this study. The design tide elevations corresponding to 1:20 year and 1:100 year return period floods have, therefore, been estimated as given in Table 7.7.

7.2.8 Design Tidal Elevations at McNutt Brook

While there has been limited documentation on the propagation of the tide from McNutt Brook to the site of the Old Board Landing Bridge, measurements taken by Dillon-Horner on April 16-17, 1980, at the Soley Brook Aboiteau upstream of McNutt Brook, suggest that the tide is amplified in the order of 0.2 m. To account for this phenomenon, the design tide elevations at Old Board Landing Bridge were reduced by 0.3 m to account for amplification between McNutt Brook and Soley Brook, and between Soley Brook and the Old Board Landing Bridge and are as indicated in Table 7.8.

TABLE 7.6
COMBINED ASTRONOMICAL AND METEOROLOGICAL TIDES, TRURO

Recurrence Interval	Elevation in Metres
1:20 year	9.9
1:100 year	10.2

TABLE 7.7
DESIGN TIDE ELEVATIONS AT BOARD LANDING BRIDGE, TRURO

Recurrence Interval	Elevation in Metres
1:20 year	10.4
1:100 year	10.7

TABLE 7.8
DESIGN TIDE ELEVATIONS AT McNUTT BROOK (1)

Recurrence Interval	Elevation in Metres
1:20 year	10.1
1:100 year	10.4

(1) In order to avoid computational problems in the model at ebb tide, the minimum tidal elevation was set to 3.0 m above mean sea level.

7.3 Design Tidal Hydrographs

7.3.1 1:20 and 1:100 Year Tides

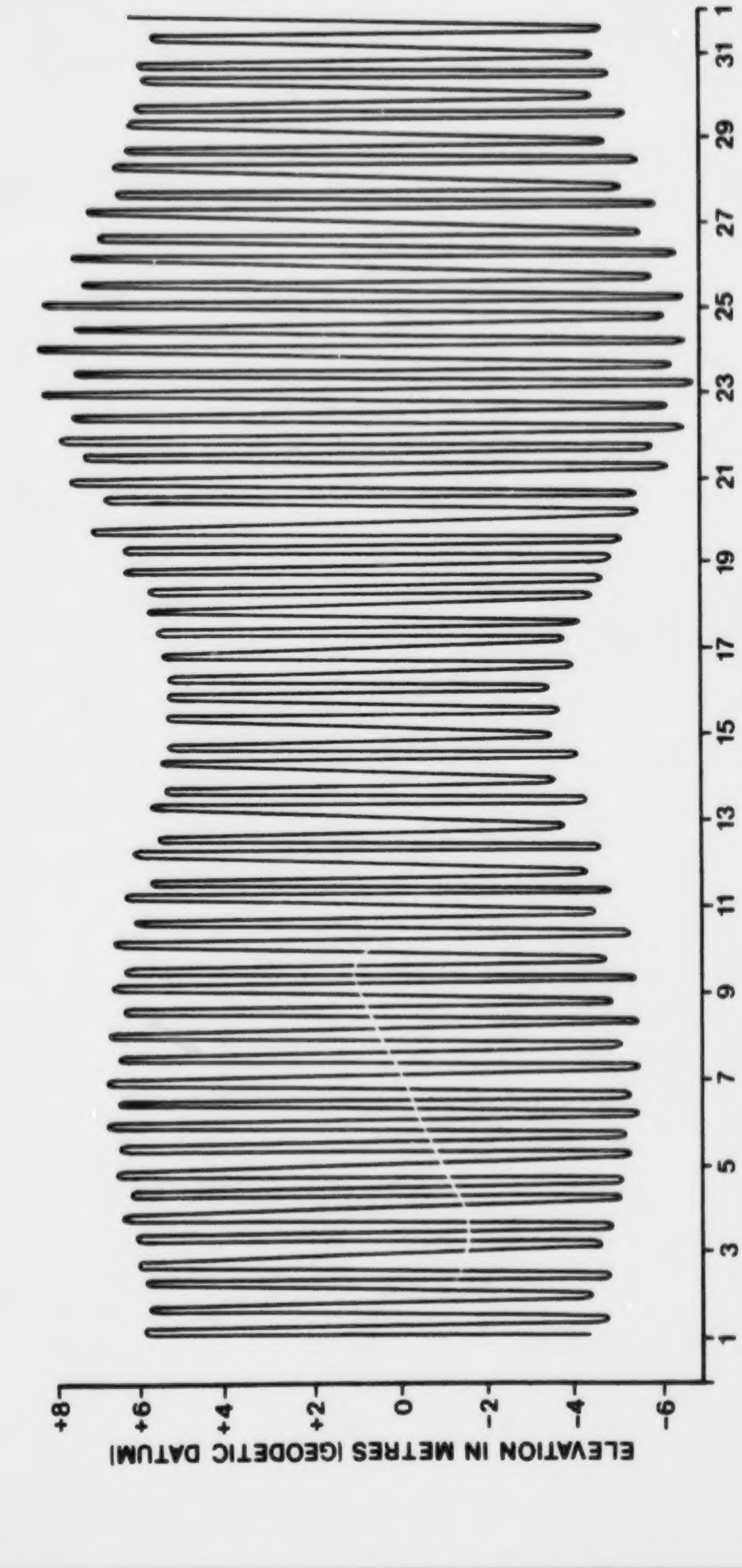
The time variation of the design tide before and after the time of occurrence of the peak magnitude is also required as input to the hydrodynamic model (over a minimum period of 72 hours). At this stage of the analyses, there were two knowns; (1) the lowest elevation of the river bed at McNutt Brook (2.1 m above mean sea level), and (2) the peak tidal elevation corresponding to each design condition (Table 7.8). With respect to the latter, it was assumed that significant meteorological disturbances would affect only one tidal peak. The peak elevation for each of the remaining tides were assumed to follow a pattern similar to that of the 18.6 year high tide period of 1976. This assumption is reasonable in view of the return period of the design conditions. In addition, in view of the relatively flashy nature of the Salmon River system, it is unlikely that lower tidal peaks on either side of the design condition would produce significantly different results in terms of the flood profile.

As indicated in Section 7.1.4, the shape of Bay of Fundy tidal hydrographs are altered from a quasi-sinusoidal curve out in the Bay to a sharp rising and gradually falling hydrograph, characteristic of the effect of bottom friction and shallow waters found in tidal estuaries as demonstrated in Figure 7.2. As discussed in Section 7.2.2 and as is evident from Figure 7.5, it is reasonable to conclude that the shape of the tide at McNutt Brook would fall somewhere between that of the Bay and that as recorded at Old Board Landing Bridge. Therefore, for the purposes of this study, it is assumed that the shape of the tidal hydrograph at McNutt Brook is best approximated by the classical sinusoidal signature of Bay of Fundy tides as recorded at Burntcoat Head. Based on these assumptions, tidal hydrographs at the downstream boundary of the study area at McNutt Brook were formulated for 1:20 year and 1:100 year return period design conditions as illustrated in Figures 7.9 and 7.10 respectively.

7.3.2 Typical Spring Tide

During the course of the study, it was decided to assess the significance of the tide as a flood causing mechanism. Conceptually, a significant runoff event combined with extreme tides, as determined in Section 7.2.8, will obviously produce flooding in the tidal estuary as the design tidal elevations alone exceed the elevation of the dyking system. It is not apparent whether or not an extreme runoff event combined with typical tidal patterns will cause flooding in the estuary. To test this case, it was decided to simulate water surface profiles in the study area corresponding to 1:20 and 1:100 year return period flood hydrographs occurring during a typical spring tide condition which has a recurrence interval once each lunar month.

As documented in Section 7.2.4, a tide with an elevation of about 8.3 m occurs on average once each lunar month at the site of the Old Board Landing Bridge. When adjusted for the effects of coastal submergence (+0.5 m), and for amplification between McNutt Brook and Old Board Landing Bridge (-0.3 m), the peak elevation associated with a typical spring tide at the downstream boundary of the study area was estimated at 8.5 m. The complete tidal hydrograph surrounding this elevation is shown on Figure 7.11.

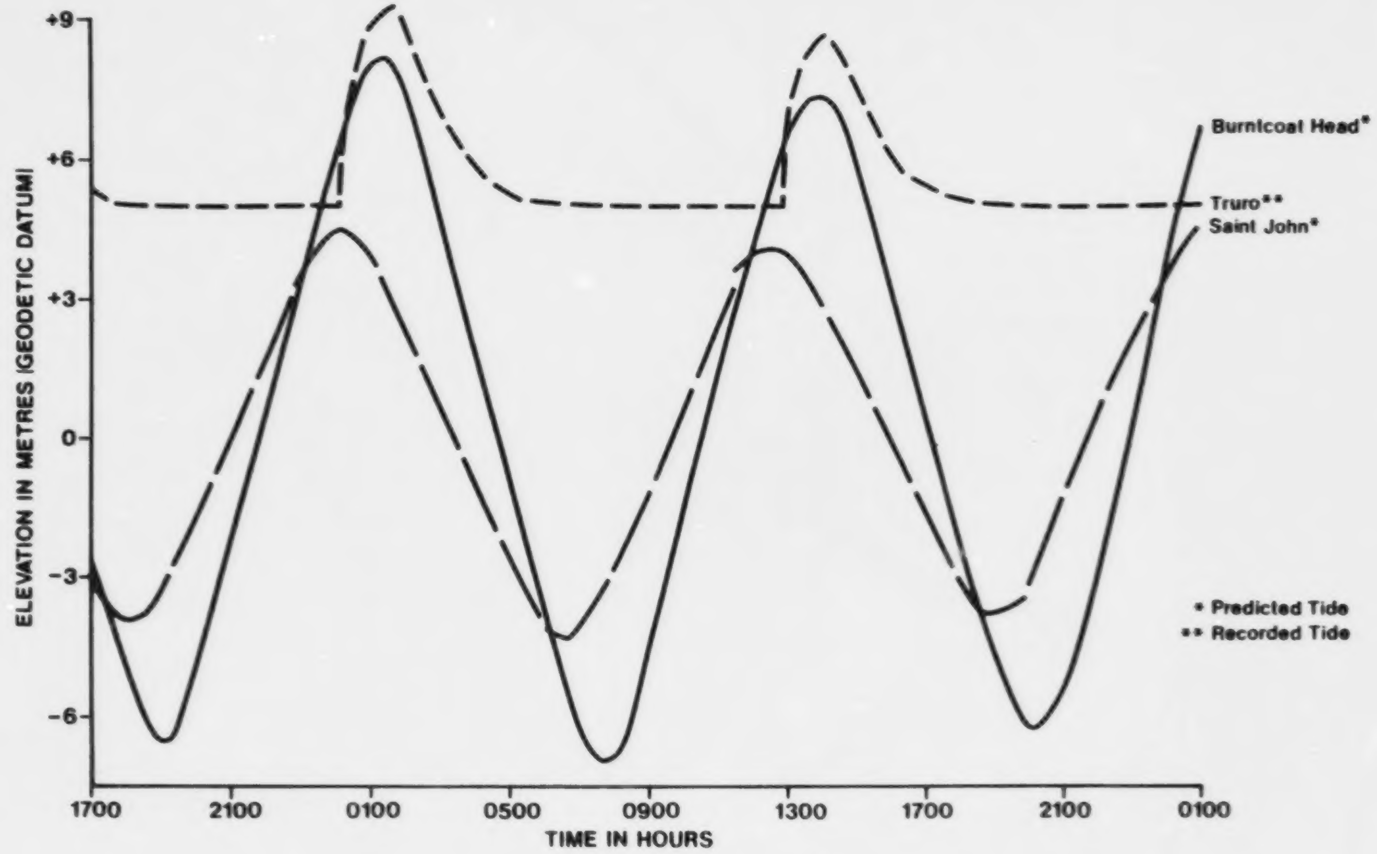


BURNTCOAT HEAD TIDES
MAY 1978
(PREDICTED)

SOURCE : CANADIAN HYDROGRAPHIC SERVICE
FISHERIES AND OCEANS CANADA

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 7.1

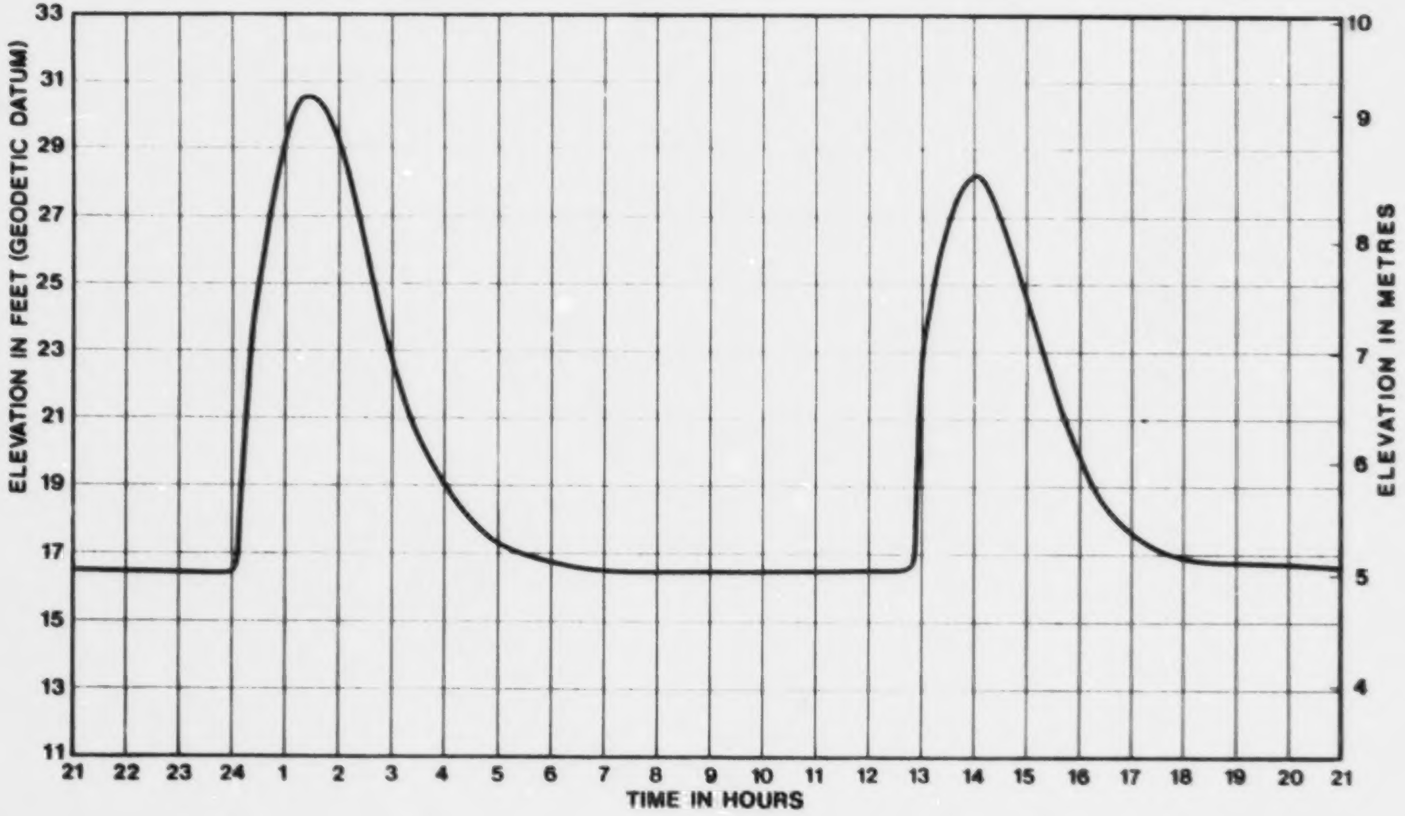


CANADA - NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

TIDAL PROPAGATION - BAY OF FUNDY

(APRIL 14-15-16, 1982)

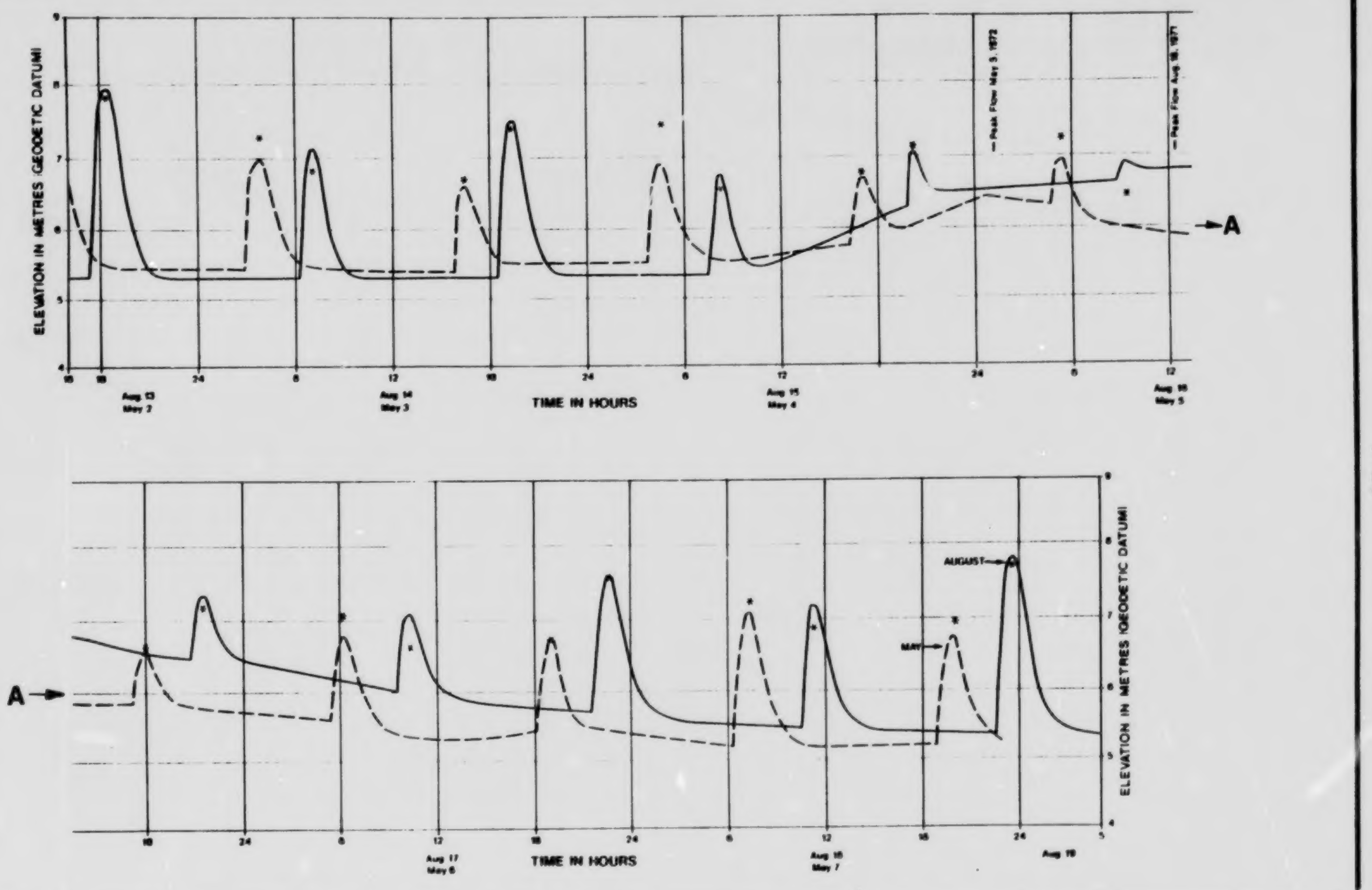
FIGURE 7.2



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

**TYPICAL TIDAL CYCLE
SALMON RIVER AT
BOARD LANDING BRIDGE**
APRIL 15, 1972

FIGURE 7.3



LEGEND

* PREDICTED TIDAL ELEVATION BASED ON RECORDED LEVELS AT SAINT JOHN

—— AUGUST 1971

– – – MAY 1972

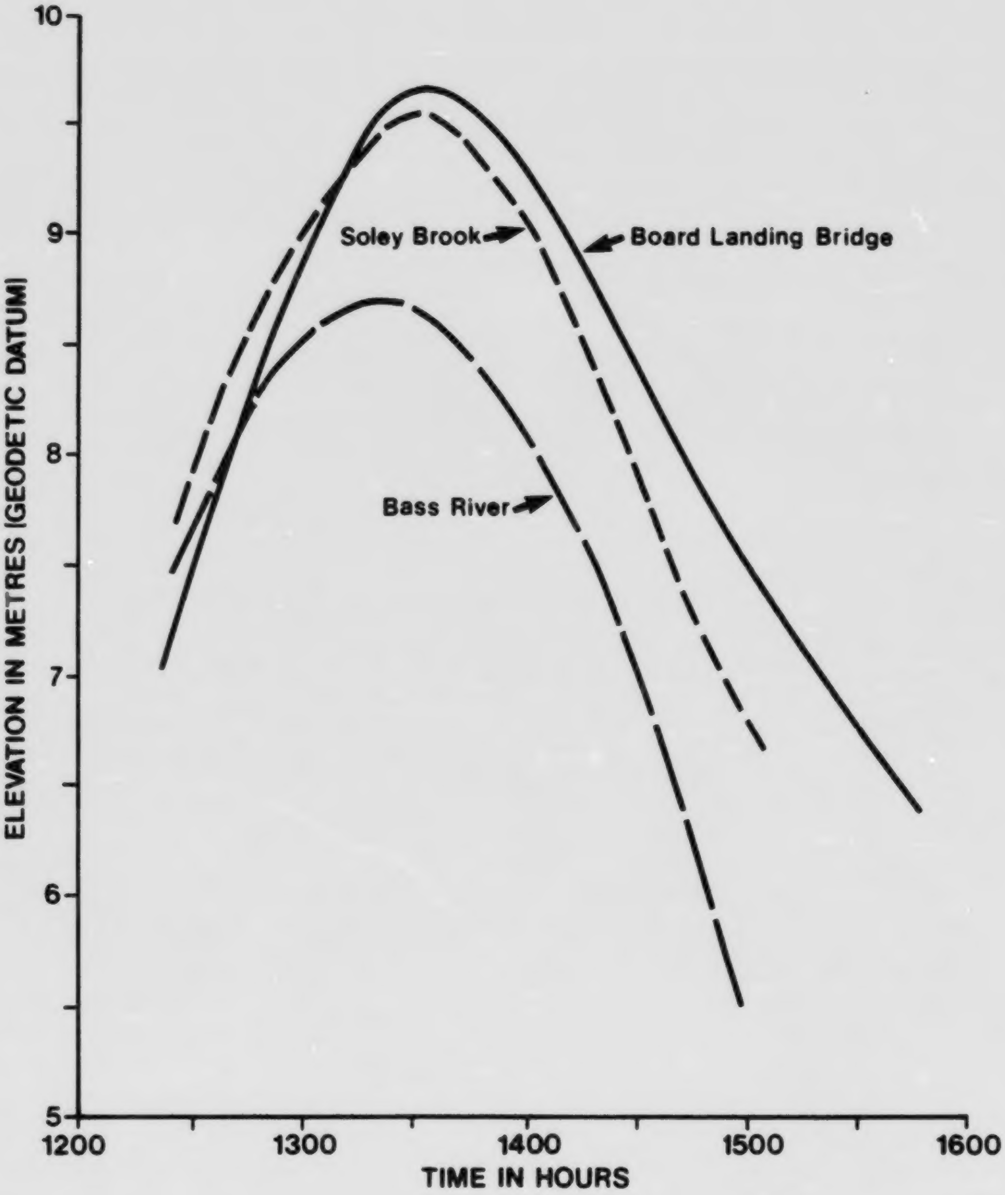
RECORDED TIDES AT BOARD LANDING BRIDGE

AUG. 13-19, 1971 AND MAY 2-7, 1972

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

TRURO HYDROTECHNICAL STUDY

FIGURE 7.4

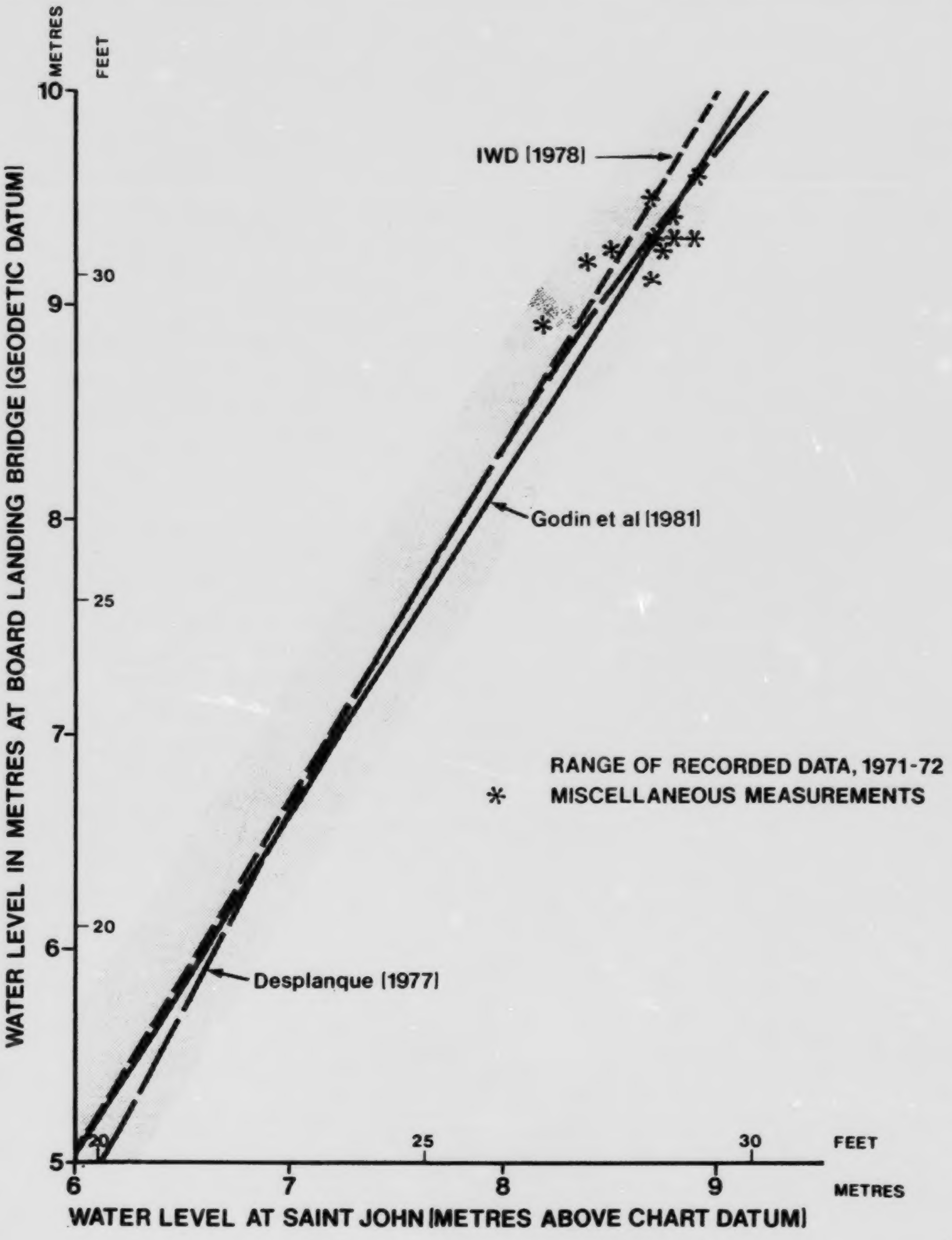


SOURCE: DILLON-HORNER, 1980

WATER LEVELS OBSERVED ON APRIL 16, 1980.

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 7.5



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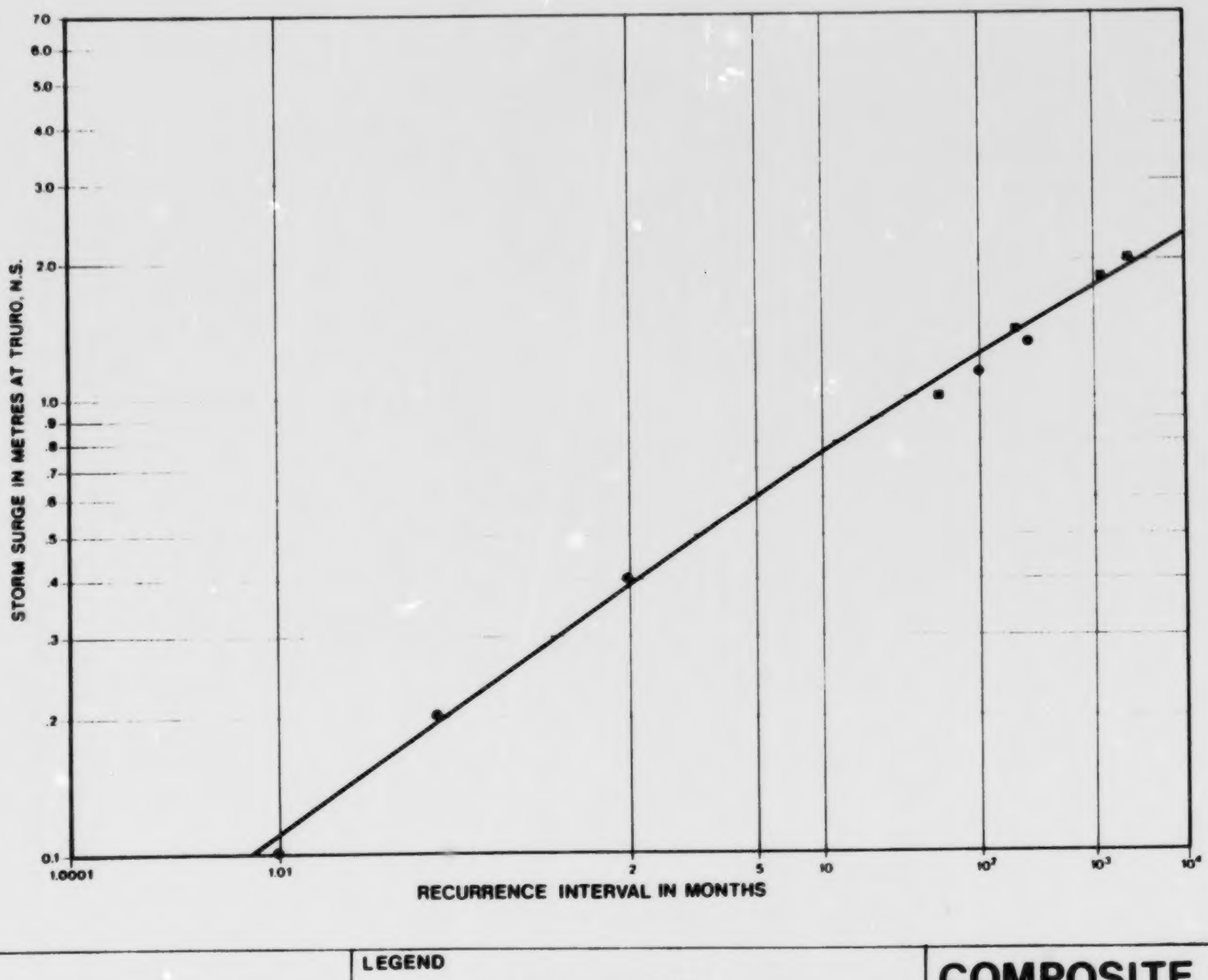
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

COMPARISON OF PREDICTION EQUATIONS TRURO VS. SAINT JOHN

FIGURE 7.6



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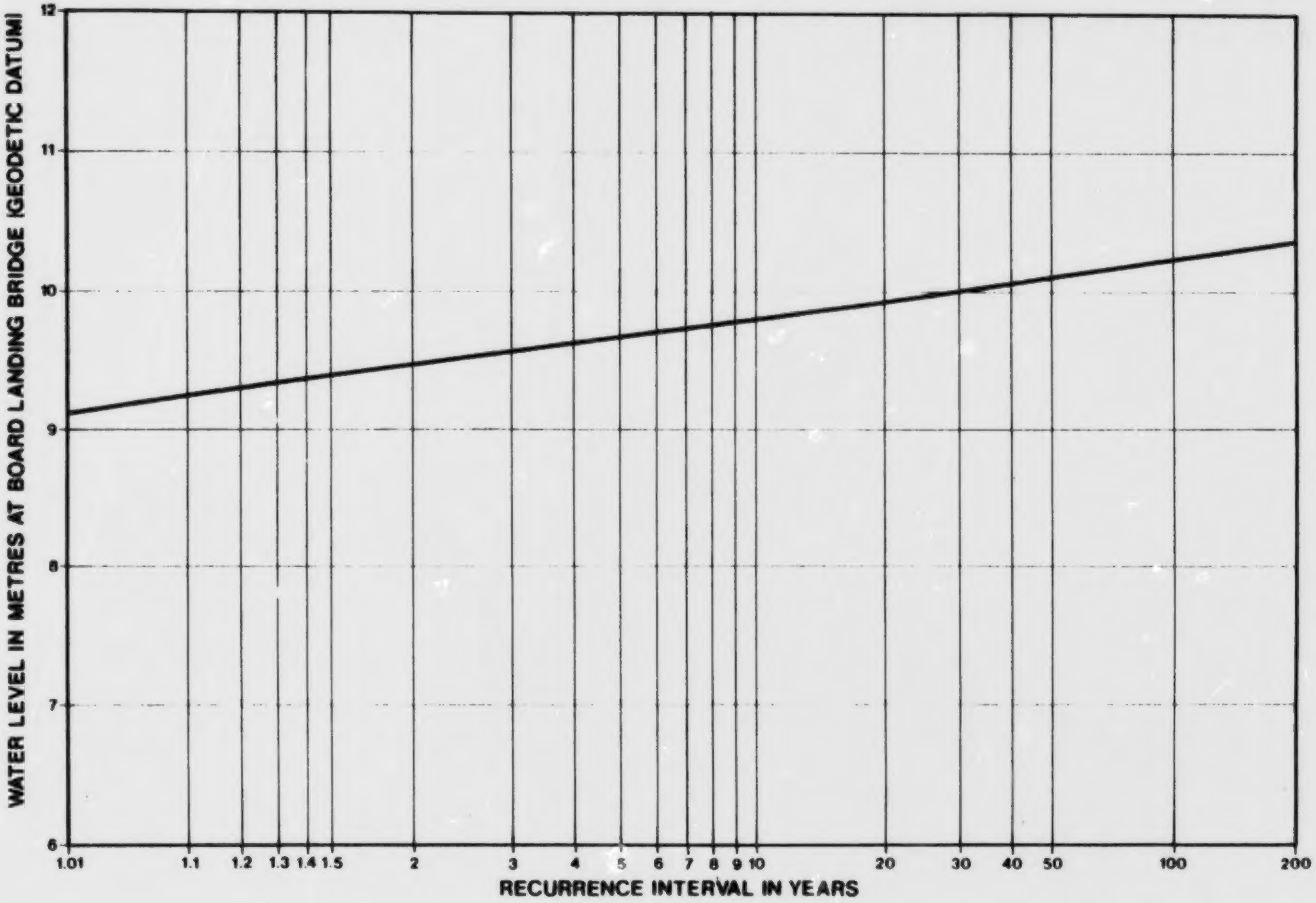
TRURO HYDROTECHNICAL STUDY

COMPOSITE STORM SURGE FREQUENCY CURVE

FIGURE 7.7



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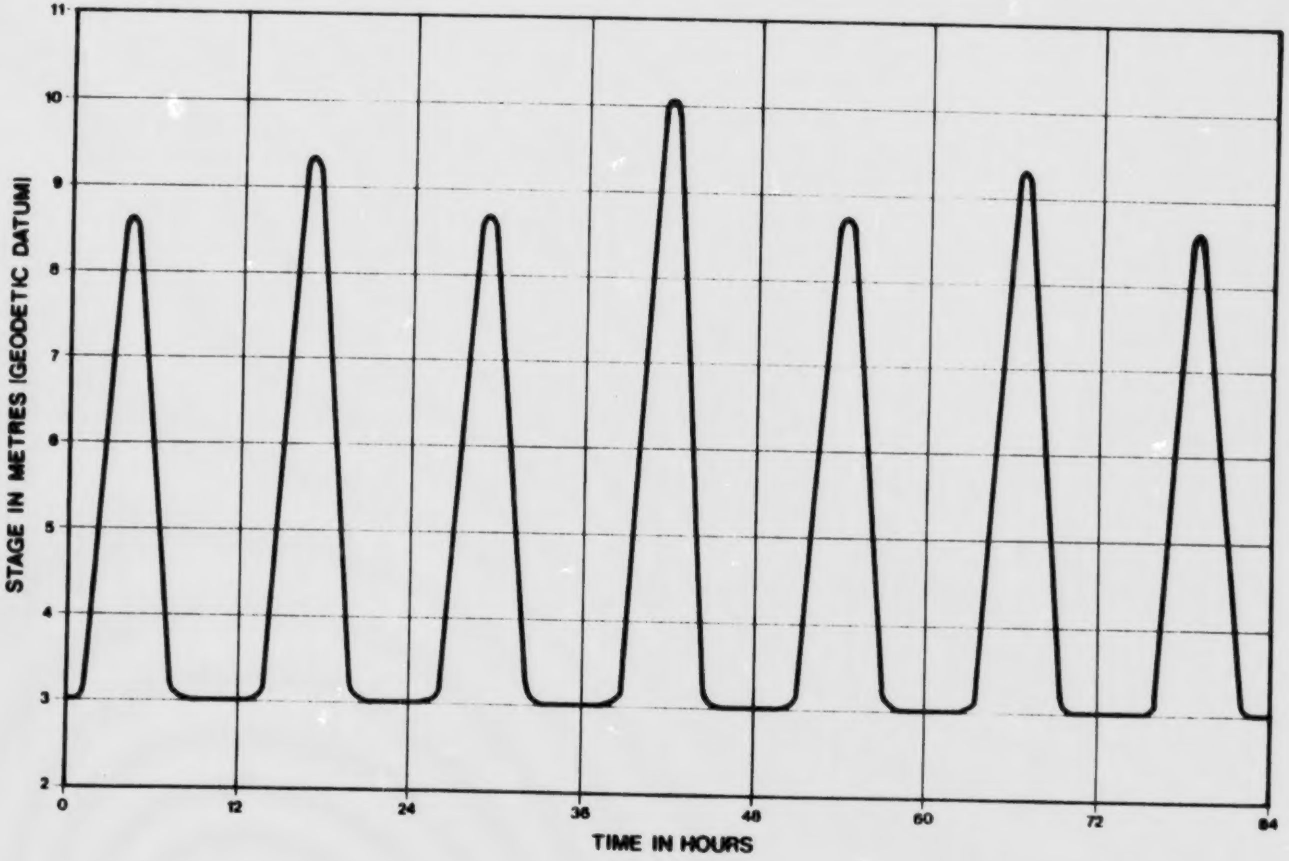
TRURO HYDROTECHNICAL STUDY

COMBINED ASTRONOMICAL AND METEOROLOGICAL TIDES - TRURO

FIGURE 7.8

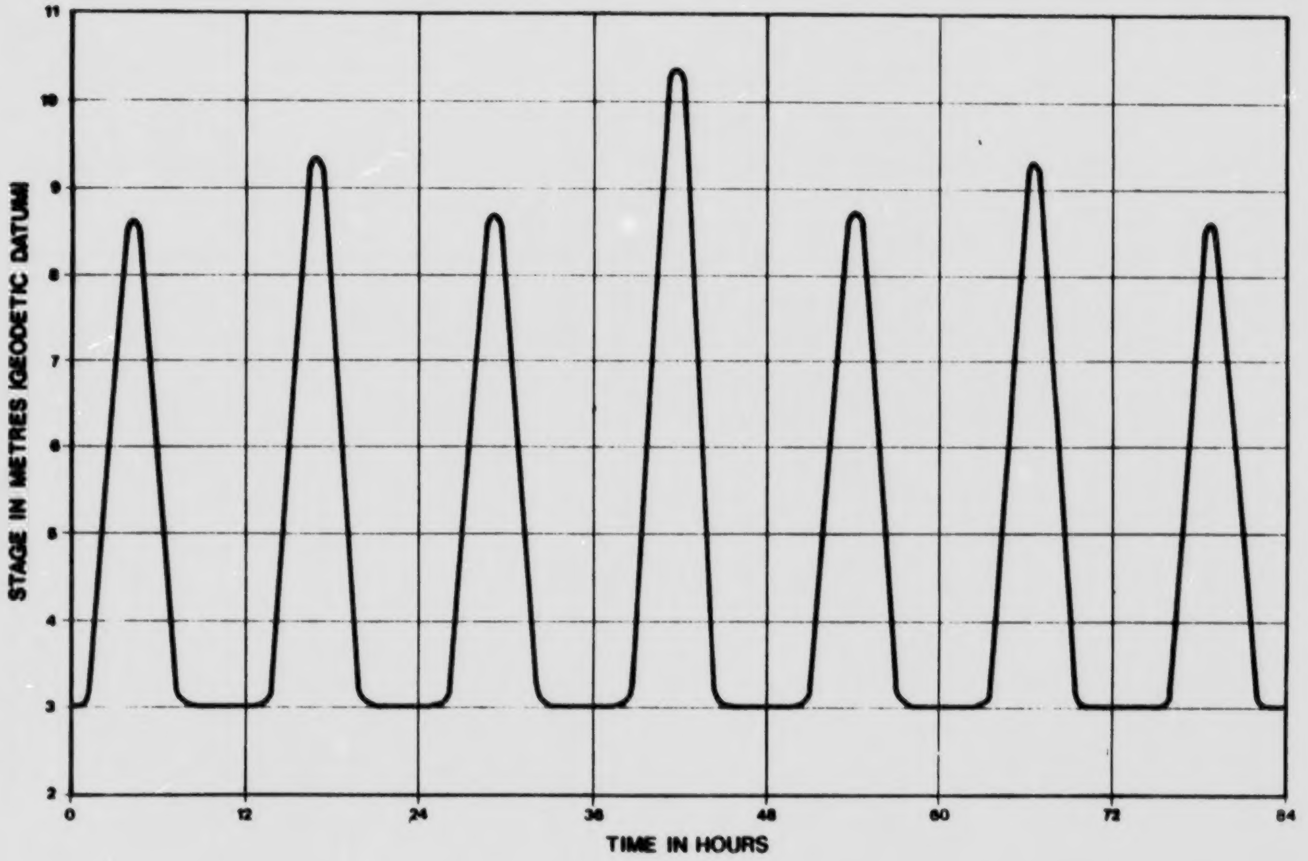


CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

1:20 YR RECURRENCE INTERVAL TIDAL HYDROGRAPH

SALMON RIVER AT McNUTT BROOK

FIGURE 7.9



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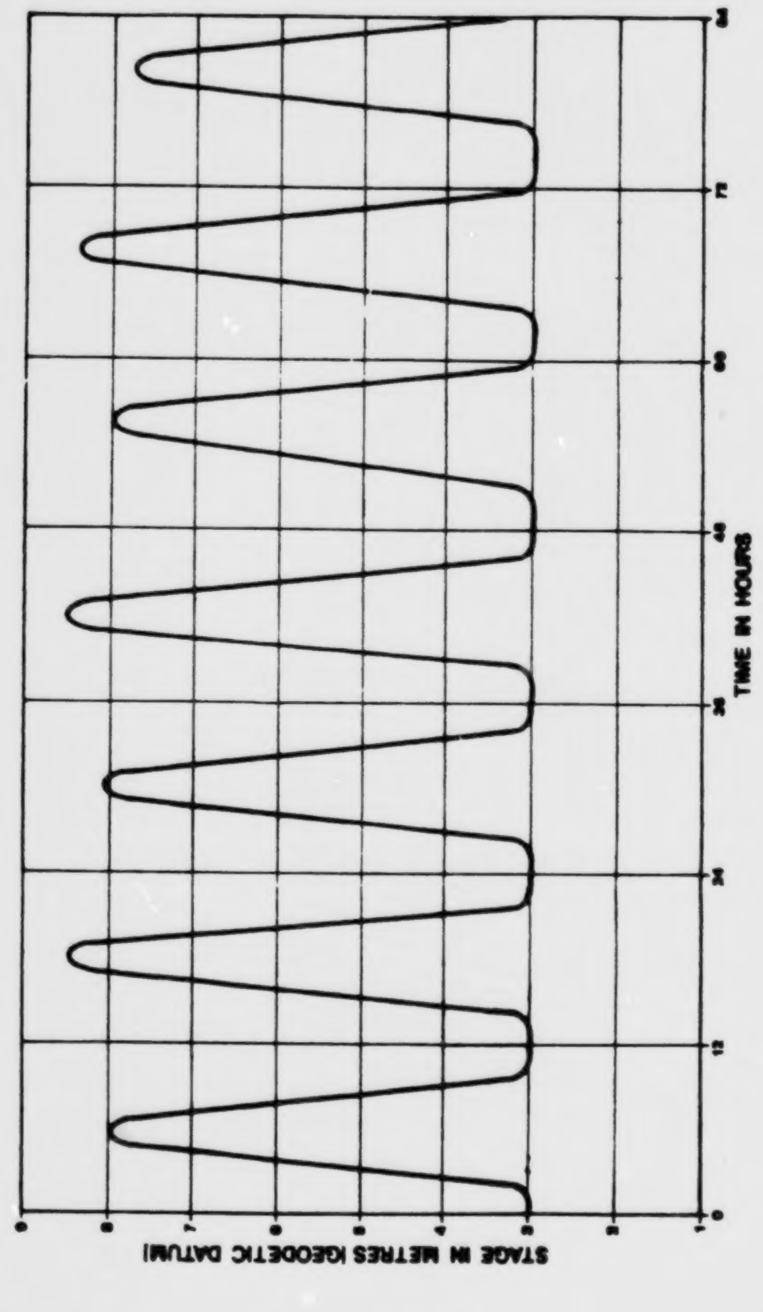
1:100 YR. RECURRENCE INTERVAL TIDAL HYDROGRAPH

SALMON RIVER AT McNUTT BROOK

FIGURE 7.10



TYPICAL SPRING TIDAL HYDROGRAPH

SALMON RIVER AT McNUTT BROOK

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 7.11

8.0 HYDRAULIC MODELLING

8.1 Hydraulic Description of the Study Area

8.1.1 General

The hydraulic analysis for the Truro area covers the stream channel and floodplain storage elements along the lower parts of the Salmon and North Rivers as shown on Figure 8.1.

The reach of the Salmon River from above Clifford Brook to the Park Street bridge has a relatively uniform steep gradient of .00232 m/m over a distance of 12.9 km (Figure 8.2). The channel is generally clean and straight with a coarse gravel bed and cohesive bank material. Numerous bedrock outcrops traverse the bed creating shallow pools and cascading flows during low to mean discharge conditions. The channel gradient sharply flattens below the Park Street bridge. Tidal silts accumulate along the lower reaches during low discharge and during high runoff conditions the silts are flushed to the wide tidal flats in Cobequid Bay.

The reach of the North River from just above the E. Lynds Bridge on Highway 311 to the Onslow Road bridge at Marshall's Corner has a relatively uniform steep gradient of .00374 m/m over a distance of 7.3 km (Figure 8.3). The channel is clean and straight with a bouldery coarse gravel bed and cohesive banks. The banks are steep and the channels are sharply incised in a relatively flat valley bottom averaging about 500 m in width. The channel banks of the North River tend to be coincident with either the left or right valley wall along its descent. Along the opposite bank on the floodplain side of the channel, there tends to be a berm of about 1 m in height, resulting from historical channelizaton projects. Occasional bedrock outcrops traverse the bed in the reaches above the Trans Canada Highway bridge. The channel bed has a flat gradient below the Onslow Road bridge at Marshall's Corner and is composed primarily of estuarine silts, confined by dykes.

8.1.2 Tides

The tidal characteristics of Cobequid Bay were described in detail in Chapter 7.0. Historical observations have indicated that the influence of tides on flooding is a predominant factor along the lower reaches of the study area.

For the purposes of this investigation, hydrodynamic simulations were undertaken for the following three downstream tidal boundary conditions:

i) 1:20 year tides at Truro
ii) 1:100 year tides at Truro
iii) typical spring tides at Truro

The magnitudes and temporal distribution of these tides are illustrated on Figures 7.9, 7.10 and 7.11. Additional details concerning the analysis and derivation of the appropriate tidal characteristics are found in Chapter 7.0.

8.1.3 Streamflow

Streamflow hydrographs were used as upstream boundary conditions on the North River, Salmon River and McClures Brook. Tributary inflows for Farnham, McCurdy, Christie, Lepper, Soley, and Higgins Brooks are represented by point lateral inflows. Other local runoff is represented as uniformly distributed lateral inflow. The development of streamflow hydrographs for the calibration runs and for the design events is described in detail in Chapter 6.0.

8.1.4 Dykes, Storage Areas and Flood Channels

Dykes have been constructed along the banks of the rivers and as discussed in Chapter 3.0, these dykes have frequently been overtopped and thus hydrodynamic modelling must be capable of simulating this phenomena. Aboiteaux are constructed in the dykes for draining freshwater flows to the Salmon River channels and preventing saline water intrusion during high tides. Modelling must, therefore, also simulate the operating characteristics of aboiteaux during flood events.

Above the Walker Street bridge on the Salmon River and the Onslow Road bridge at Marshall's Corner on the North River the channel and floodplains have steep gradients with little natural storage. When the main channel banks are overtopped, the time of storage is short because of the steep gradient in the main channel.

Below the Walker Street bridge and the Onslow Road bridge, the existing dyke system would overtop at high flood stages. Movement of water behind the dykes would then occur to pass flow from the overtopping points to downstream points in the main channel of the Salmon River.

8.1.5 Other Hydraulic Factors

The influence of ice jams (both freshwater jams upstream and downstream and tidal ice in the estuarine portion) on flood levels was discussed previously in Chapters 3.0 and 4.0. It was considered that insufficient data were available for the purpose of detailed modelling of ice jam flood characteristics. However, some sensitivity analyses were undertaken as described in Section 8.6.

The Salmon River estuary is also generally subjected to significant erosion and sedimentation. Cross-section surveys at different times of the year indicate that the bottom bed elevation changes as much as 1 m during a normal annual cycle of flow and tide conditions. In fact, variations of up to 2-3 m have been observed over several years (see Chapter 3.0). Therefore, a degree of uncertainty arising from the unpredictable bed elevation changes had to be incorporated in the application and evaluation of the hydrodynamic model.

8.2 Model Review and Selection

8.2.1 Models Considered

Several hydraulic models were considered in 1983 for application in this project. The main criteria for selection of an appropriate model are summarized as follows:

1. model reliability
2. model flexibility
3. preferably non-proprietary
4. capabilities to handle special hydraulic conditions in Truro
5. minimal cost of running
6. expertise available

In selecting a model, significant consideration was given to the special requirements for this project. In particular, it was necessary that the capability of the model included the following factors:

1. downstream tidal boundary conditions
2. upstream discharge boundary conditions
3. joint occurrence of tides, flood discharge and overtopping of dykes
4. channel and floodplain storage
5. operation of aboiteaux in the dykes
6. flow in flood channels behind the dykes from several overtopping locations along the dyke system to downstream points
7. other hydraulic factors including structures; head losses due to channel and floodplain conditions; and bed conditions

8.2.2 Model Selection

The large tidal variations and significant channel and floodplain storage effects precluded the use of a steady-state hydraulic model such as HEC-2. The use of dynamic streamflow routing linked with the HEC-2 steady state model was also considered to be unsuitable since a steady-state analysis in conjunction with a hydrodynamic analysis creates problems in defining the link, both temporally and spatially, between the two analyses. In addition, the overtopping of dykes during flood events requires a dynamic analysis to assure accurate calculation of maximum stage elevations in the flood channels.

After consideration of a number of models, it was concluded that application of either the 1-D or DWOPER models would be most suitable for the purposes of this investigation. This conclusion is consistent with the findings of other similar investigations (e.g. MacLaren Atlantic, 1979).

The 1-D model and the DWOPER model are capable of simulating a transient flow regime in channels which may consist of embayment storage areas, various hydraulic control structures and tidal boundary conditions, in addition to other hydraulic factors listed in the previous section.

The Environment Canada One Dimensional (1-D) hydrodynamic model is based on theory developed at M.I.T. in the early 1970's (Harleman and Lee, 1967). The model evolved in Canada from its initial coding and application to the St. Lawrence River Water Quality study in 1973 to recent applications on the Peace-Athabasca Delta.

The ability of the 1-D model to handle the following factors made its use more attractive than the DWOPER model (1):

(1) Recent versions of the DWOPER model have included some of these capabilities.

1. capability to handle second and higher order tributaries
2. ability to simulate more than two dykes overtopping points and aboiteaux
3. ability to model flood channels with return flow loops to the main channels.

In 1982, neither the DWOPER nor the original 1-D models were suitable for cases where the flow depth was negligible or zero due to complete breakdown of the momentum equation. However, this capability was added to the 1-D model for the Truro and Peace-Athabasca Delta applications.

Finally, while the DWOPER model had been successfully used by others in several previous hydrotechnical studies (Perks, 1983, MacLaren Atlantic, 1979), the study team for this project was most familiar with the 1-D model, having applied it successfully in a number of previous investigations. Therefore, it was concluded that with previous experience in modifying and developing the Environment Canada One-Dimensional (1-D) Hydrodynamic model to suit various individual projects, this same model could also be successfully applied to the Truro project.

8.2.3 Description of 1-D Hydrodynamic Model

The basic equations describing the process of flood wave propagation in open channels are the Saint Venant equations. These equations represent the conservation of mass and momentum of flow of a fluid in the channel.

Derivations of the Saint Venant equations by the "material" method were made by Harleman and Lee (1967). The equations and a discussion of the implicit finite difference solution techniques are given in Appendix C and described in more detail in the user's manual and associated documentation (Env. Can. 1982, Env. Can. 1983, Env. Can. 1987, Env. Can. 1986).

1. irregular cross-sectional geometry
2. linear interpolation and extrapolation of cross-sections between input cross-sections
3. two types of physical off-channel storage
    i) shoal-area storage, used to differentiate the passive part of the cross-section with the conveyance area. In this case, the core (or conveyance) area is defined by the user
    ii) embayment storage, used to represent storage in small and shallow bays or lakes
4. Manning's coefficient of roughness, which may vary with water surface elevation, and from cross-section to cross-section
5. capability to compute steady-state conditions that can be used as initial conditions
6. time-dependent lateral inflow or outflow
7. space increment that can vary from one reach to another
8. any number of reaches connected to a node (the present model setting is for a maximum of 5 reaches per node, but this can be increased to any value by changing the dimensions of arrays and matrices)
9. friction variation caused by full or partial ice cover, and by ice cover growth or decay
10. reduction in cross-sectional area caused by ice thickness
11. flow overtopping roads, dykes or embankments adjacent to a reach, and/or through culverts or aboiteaux
12. delineation of flood channels in the floodplain
13. indirect method to respond to channels that occasionally dry up or are subject to very low flows
14. internal boundary conditions, such as small rapids or falls, bridges, weirs, gates and spillways, represented by a relationship of a family of curves between water elevations (upstream and downstream of the structure) and discharges
15. external boundary conditions such as discharge or water surface elevation hydrographs. In the case of downstream boundaries, a relationship between water levels and discharges representing natural controls or hydraulic structures can be

> used. The model also has the capability to compute an approximate stage-discharge relationship for downstream boundaries, according to the physical characteristics of the reach in the vicinity of the boundary.

The river system is represented by a network of river reaches interconnected by nodes. The network schematization is defined by the user in order to represent the natural system but with a minimum number of reaches and nodes to avoid unwarranted computational expenses.

Each reach is described by the following characteristics:

1. a length
2. one or many Manning's coefficients of roughness
3. dividing the reach into a number of mesh points for computational purposes
4. a space increment
5. a table of hydraulic parameters (total top width, core top width, core area, wetted perimeter, and total area, at different water depth or elevation) for each input cross-section.

One constraining feature of the model is that a minimum of 4 space increments (5 mesh points) must be used for each reach. It is recommended to always have an even number of space increments (odd number of mesh points), in order to limit the transfer from reach to reach of numerical solution differences caused by input errors or internal boundary calculations.

The hydraulic parameter tables can be manually generated. However, front-end computer progams such as COORD1, COORD2 (Env. Can. 1984), have been written in order to generate these tables directly from input cross-sectional data. These programs allow the data to be read from processed hydrographic charts, digitized survey field sheets, topographic maps and aerial survey data bases.

Initial and external boundary conditions must be specified in order to run the model. The external boundary conditions represent the hydraulic behaviour at the extremities (upstream, downstream) of the network, during the simulation period. They are represented by elevations or discharge observations and measurements, and must fulfill the continuity flow requirements.

To adequately simulate the flow regime in Truro, a BREACH subroutine was developed and incorporated in the model. The key parameters in the BREACH model are the dyke elevation and the crest length for which overtopping occurs. The elevation of the remainder of the dyke is considered in the model to extend vertically in order to maintain appropriate model representation at high flood stages. The flood channels in the network must maintain a minimal amount of water during simulation for periods of no overflow from the main channels or for periods when the water levels in the downstream channels become very low. This was accomplished by having an artifically lower bottom elevation in the flood channel to create a small "notch". This modification resulted in no change to the cross-sectional area and the wetted perimeter of the channel in the active flow range. The BREACH subroutine is described in detail in the associated model documentation (Env. Can., 1986).

8.3 Model Development

8.3.1 Model Schematization

The model schematization, which was derived is illustrated on Figure 8.1, and consists of a total of 26 reaches. Each reach was described by a minimum of three to as many as nine surveyed cross-sections depending on the variation of channel characteristics. The location of surveyed cross-sections depended on significant changes in channel gradient and area, the presence of bridges and pipeline crossings, and the existence of embayment storage areas in the case of flood channels. (The field surveys and cross-section locations are discussed in Chapter 5.0 and the cross-sections input to the model are reproduced in Appendix A).

Adjacent reaches are connected by upstream and downstream nodes. The location of nodes was determined based on connectivity requirements, breach requirements, discretization criteria and location of structures in the network. The 1-D model represents bridges according to the cross-sectional characteristics which are input at the bridge location.

Nodes defining the model reaches were selected at tributary confluence points, and as much as possible at bridges where flow constriction would be possible at high discharge. However, it was not always practical to locate bridges at the end of reaches due to the need to keep reach lengths greater than a minimum value in accordance with the computational unit time step. For example, the Stewart Bridge, the Trans Canada Highway Bridge, the Murray Bridge, the CNR Bridge on the Salmon River and the Park Street Bridge are all located at nodes and, therefore, the surveyed cross-sections at these structures are used directly in the model computations. The CNR Bridge on the North River and the Marshall's Corner Bridge fall on mesh points and, therefore, interpolation of cross-sectional area was avoided. It was not possible to locate the Valley Bridge, the Highway 311 Bridge and the Highway 102 Bridge at either a node or a mesh point and hence the model interpolates cross-sectional properties at these locations.

A complex hydraulic condition was encountered just downstream of node 25 (CNR Bridge on the Salmon River). A watermain crossing is located about 222 m from the CNR Bridge, and the Walker Street Bridge is located approximately 24 m further downstream. There were two cross-sections surveyed at the site of the CNR Bridge; one immediately upstream and one immediately downstream. For modelling purposes, the first one was used as the last cross-section for Reach 140 and the first cross-section for Reach 150; while the second one was assumed to represent conditions about 30 m downstream of the bridge and formed the second cross-section for Reach 150. The surveyed cross-section at the watermain crossing formed the third cross-section for Reach 150. The next surveyed cross-section, at about 271 m, is just downstream of the Walker Street Bridge, formed the fourth cross-section for Reach 150. Since a mesh point spacing of about 148 m was found necessary for Reach 150 (due to the requirement for a minimum of five mesh points), neither the watermain crossing nor the Walker Street Bridge was exactly at a mesh point, although the schematization is considered to be an adequate representation during flood conditions.

A tabular description of the schematization of the network (as shown in Figure 8.1) is summarized in Tables 8.1 and 8.2. Additional details of the spatial discretization of all the reaches are summarized in Table 8.3.

In the model configuration a starting (upstream) node and a finishing (downstream) node are boundary conditions. Input between nodes is provided as lateral inflow (see Section 8.1.3).

The downstream node 28 is represented by a tidal boundary condition (Z(t)) (see Section 8.1.2). The upstream nodes, 1 and 2 for North River, 23 for Salmon River and 18 for McClures Brook have hydrographs (Q(t)) as boundary conditions (see Section 8.1.3).

The network schematization also includes 11 breach locations as indicated on Figure 8.1. This permits simulations of flows over dykes, flows through aboiteaux or culverts, and combinations of both. The direction of flow is two-way in the cases of dykes and culverts, and is one way in the case of aboiteaux. A summary description of all the breach structures is given in Table 8.4.

The actual crest lengths for dyke overtopping are very difficult to determine and undoubtedly vary according to flood stages on either side of the dyke. Initial values for crest lengths were provided by technical committee members who had observed overtopping of the

TABLE 8.1

TRURO MODEL

REACH NODE CONNECTIVITY DESCRIPTION

Reach No.	Reach Description	Upstream Node No.	Down-stream Node No.	Reach Length (m)
010	South Branch North River upstream end	2	3	1340
030	West North River, upstream end	1	3	771
040	North River, confluence of North River to Stewart Bridge	3	4	1365
050	North River, from Stewart Bridge to Hwy. 104 Bridge	4	5	2923
060	North River, from Hwy. 104 Bridge to north of CNR	5	6	1049
070	North River, from north of CNR to Salmon River	6	27	1581
080	Flood Channel, west of North River from north of Onslow Road Bridge at Marshall's Corner on North River to CNR embankment	6	7	762
090	Flood Channel, east of North River from north of Onslow Road Bridge at Marshall's Corner to CNR embankment at Farnham Brook	6	10	1186
100	Flood Channel, Farnham Brook from CNR embankment to Salmon River	11	13	707
110	Flood Channel, north of Salmon River from CNR embankment near North River to McCurdy Brook near Salmon River	8	9	2388
120	Salmon River, upstream end from above Clifford Brook to Christie Brook	23	24	3998
130	Salmon River, from Christie Brook to Murray Siding Bridge	24	29	2294
140	Salmon River, from Murray Siding Bridge to Lepper Brook	29	25	4863

TABLE 8.1 (cont'd)

Reach No.	Reach Description	Upstream Node No.	Down-stream Node No.	Reach Length (m)
150	Salmon River, from Lepper Brook to Park St. Bridge	25	26	1776
160	Salmon River, from Park St. bridge to North River	26	27	1338
170	Flood Channel, north of Salmon R. from Lepper Bk to Park St. Bridge	25	26	1018
180	Flood Channel, north of Salmon R. from Park St. bridge to Farnham Brook near Salmon River	12	13	956
190	Flood Channel, south of Salmon R. from Park St. bridge to 762 m south of confluence of Salmon and North Rivers	14	16	977
200	Flood Channel, south of Salmon R. from confluence of Salmon & North Rivers going 762 m south	15	16	759
210	Flood Channel, south of Salmon R. from 762 m of confluence of Salmon & North Rivers to Hwy. 102 viaduct	16	17	2049
220	Flood Channel, south of Salmon R. from Hwy. 102 viaduct to McClures Bk	17	21	357
230	McClures Brook reach above culvert (railway crossing bridge)	18	19	296
240	McClures Brook from Hwy. 102 to culvert above Doggett Brook	20	21	2205
250	McClures Brook, from aboiteau on Salmon River to Hwy. 102	21	22	774
260	Flood Channel, south of Salmon R. from Hwy. 102 near Salmon River to McClures Brook at Salmon River	17	22	396
270	Salmon River, from confluence with North River to McNutt Brook	27	28	7495

TABLE 8.2

TRURO MODEL

BREACH CONNECTIVITY DESCRIPTION

Breach No.	Breach Description	From (1) Reach at No.	(m) (2)	To (1) Reach at No.	(m) (2)
1	Farnham Brook Culvert on CNR on east side of North R.	90	1087	100	88
2	Railroad Culvert on west side of North River	80	699	110	149
3	Farnham Brook aboiteau to Salmon River	100	619	160	836
4	McCurdy Brook aboiteau to Salmon River	110	2239	270	460
5	Railroad Crossing on McClures Brook	230	259	240	184
6	McClures Brook aboiteau to Salmon River	250	678	270	3595
7	Upstream of Park St. Bridge overtopping to north	150	1702	180	60
8	Downstream of Park St. Bridge overtopping to south	160	167	190	61
9	Confluence of North and Salmon Rivers overtopping	160	1171	200	95
10	Upstream of CNR Bridge overtopping to north	140	4769	170	64
11	Upstream of Hwy. 102 overtopping to south	270	2902	210	1878

(1) See Table 8.3 for locations relative to mesh point distances

(2) Measured from the upstream end of the reach.

TABLE 8.3
IDENTIFICATION OF MESH POINTS FOR EACH REACH

Reach No.	Length (m)	X (m)	Distance of Mesh Points from end of Reach (m) (1)*	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
10	1339.6	167.4	0	167.4	334.8	502.4	669.8	837.2	1004.4	1171.8	1339.6										
30	771.3	128.6	0	128.6	257.0	385.7	514.3	642.7	771.3												
40	1365.8	136.6	0	136.6	273.2	409.8	546.3	682.9	819.5	956.1	1092.7	1229.3	1365.8								
50	3922.9	163.4	0	163.4	326.8	490.2	653.7	817.8	980.8	1144.2	1283.2	1471.3	1634.5	1797.9	1961.3	2125.0	2288.4	2451.8	2615.2	2778.7	2942.1
60	1049.4	175.0	0	175.0	349.7	524.7	699.7	874.4	1049.4							3105.5	3268.9	3432.6	3596.0	3759.5	3922.9**
70	1580.8	131.7	0	131.7	263.4	395.1	526.8	658.5	790.2	922.3	1054.0	1185.7	1317.4	1449.1	1580.8						
80	762.2	127.1	0	127.1	254.0	381.1	508.2	635.1	762.2												
90	1186.0	197.6	0	197.6	395.4	593,0	790.5	988.4	1186.0												
100	707.3	176.8	0	176.8	353.7	532.9	707.3														
110	2388.1	298.5	0	298.5	596.9	895.4	1194.2	1492.7	1791.2	2089.6	2388.1										
120	3997.6	249.9	0	249.9	499.7	749.6	999.4	1249.1	1499.1	1748.8	1998.8	2248.8	2498.5	2748.5	2998.2	3247.9	3498.5	3747.6	3997.6		
130	2293.6	191.2	0	191.2	382.3	573.5	764.6	955.8	1146.6	1337.8	1529.0	1720.1	1911.3	2102.4	2293.6						
140	4862.5	186.9	0	186.9	374.1	561.0	748.2	935.1	1122.3	1309.2	1496.0	1683.2	1870.1	2057.3	2244.2	2431.1	2618.3	2805.2	2992.4	3179.3	3366.5**
150	1775.9	147.9	0	147.9	296.0	444.0	592.1	739.9	887.8	1036.0	1183.8	1332.0	1479.9	1628.1	1775.9						3556.4
160	1338.1	167.3	0	167.3	334.5	501.8	668.9	836.3	1003.7	1170.7	1338.1										3740.2
170	1018.3	127.1	0	127.1	254.6	381.7	509.2	636.3	763.7	890.8	1018.3										3927.4
180	955.8	119.5	0	119.5	239.0	358.5	478.1	597.3	716.8	836.3	955.8										4114.3
190	977.4	122.2	0	122.2	244.4	366.5	488.7	611.9	733.1	855.2	977.4										4301.5
200	759.1	189.6	0	189.6	379.6	569.2	759.1														4488.4
210	2048.8	341.5	0	341.5	682.9	1024.4	1365.9	1707.3	2048.8												4675.6
220	356.7	89.0	0	89.0	178.4	267.7	356.7														4862.5
230	295.7	73.8	0	73.8	147.9	221.9	295.7														
240	2204.9	367.4	0	367.4	735.1	1102.4	1469.8	1837.5	2204.9												
250	774.4	193.6	0	193.6	387.2	580.8	774.4														
260	396.3	99.1	0	99.1	198.2	297.3	396.3														
270	7495.1	187.5	0	187.5	374.7	562.2	749.4	936.9	1124.4	1311.6	1499.1	1686.3	1873.8	2061.3	2248.5	2436.0	2623.2	2810.7	2998.2	3185.4	3372.9**
				3560.1	3747.6	3935.1	4122.3	4309.8	4496.9	4684.4	4872.0	5059.2	5246.7	5433.8	5621.3	5808.8	5996.0	6183.5	6370.7	6558.2	6745.7
				6932.9	7120.4	7307.6	7495.1														

* Mesh Point numbers

** Additional mesh points for this reach; numbering continues sequentially

TABLE 8.4

Breach Dimensions and Parameter Values Used in Breach Routine***

Breach Number	1	2	3	4	5	6	7	8	9	10	11
Reach number upstream	90	80	100.	110.	230.	250.	150.	160.	160.	140.	270.
Distance from u/s node (m)	1087	699	619	2239	259	678	1702	167	1171	4769	2902
Reach number downstream	100	110	160	270	240	270	180	190	200	170	210
Type*	1	1	2	2	1	2	3	3	3	3	3
Shape	rect.	circ.	rect.	circ.	rect.	circ.					
Discharge coefficient **	0.95	0.95	0.95	0.95	0.95	0.95	-	-	-	-	-
Diameter or width (m)	8.35	2.13	3.2	1.8(x3)	9.15	1.8(x3)	-	-	-	-	-
Length (m)	12.2	9.15	23.8	18.9	7.6	37.2					
Height (m)	2.43	-	0.76	-	3.35	-					
U/S invert elevation (m)	7.9	8.0	6.7	6.1	9.5	3.7					
D/S invert (m)	6.9	7.7	6.5	6.0	9.3	3.4					
U/S crown elevation (m)	10.4	10.2	7.4	7.9	12.8	5.5					
Manning's n	.012	.024	0.012	.012	.012	.012					
Dyke crest elevation (m)	12.4	12.7	9.4	10.1	13.6	9.9	9.6	10.5	9.9	14.3	9.9
Dyke crest length (m)	9.1	6.1	152	152	61	305	122	101	227	61	1067

* Type No. 1 = culvert with dyke
2 = aboiteau with dyke
3 = dyke only

** Function of channel contraction and entrance geometry

*** Model dimensions may differ slightly from actual measurements due to modelling assumptions dykes during past flood events. In the course of actual simulations, the crest lengths were slightly adjusted in order to render smooth hydraulic transient responses over the structures. These changes might have some effect on the duration of overflows but would not significantly affect the flow volumes.

In modelling the bank overtopping along the Salmon River, it was desirable to locate the breach where the overtopping first occurs. The choice of top-of-dyke level also reflects the mean elevation of the section of dyke under consideration. Overtopping on the south bank is accommodated immediately downstream of the Park Street bridge by Breach 8 and at the confluence by Breach 9. From the confluence to the Highway 102 Bridge on the Salmon River, overtopping of the dyke on the south bank is modelled by Breach 11. At the Park Street Bridge, overtopping on the north bank is accommodated by Breach 7 over Park Street.

Flood channel 210 allows flood overflows originating from the Park Street bridge area, at the confluence of the Salmon and North Rivers, and at Breach 11 upstream of the Salmon River bridge to be routed directly to the Highway 102 viaduct at node 17. At this point, these flows can either pass through the viaduct and/or progress to the Robie Street underpass and/or the McClures Brook bridge at node 21. Reach 260 includes storage adjacent to the dyke from the viaduct downstream to McNutt Brook.

Although the dykes have overtopped in the past along the north bank of the Salmon River (Reach 270), no storage zones or flood channels were included in the Higgins Brook area. The available storage was believed to be too small in comparison to tidal volumes to have any effect on computed stages. For the purposes of modelling, it was assumed that Soley Brook is intercepted by flood channel R250 and then discharges into the Salmon River through Breach 6 instead of discharging directly to the Salmon River.

8.3.2 Hydraulic Properties

The cross-sections incorporated in the model are given in Appendix A. Cross-sections incorporating surveyed channel sections and mapped floodplains were derived using the following criteria:

1. cross-section locations, identified on the 1977 aerial photography of scale 1:5472, were transferred to the 1:1200 scale and 1:2400 0.5 m contour topographic maps. The horizontal location root mean square error in this transfer was estimated to be equal or less than ±5 m.
2. the surveyed cross-sections were extended from low relief or flat terrain identifiable on the 1:1200 and 1:2400 scale maps, on the left bank, to identifiable low relief terrain on the right bank.
3. the cross-section data were then recorded on a coding sheet for the COORD-1 input program. The data for each cross-section was arranged starting from the left extremity of the cross-section and proceeding in a continuous fashion to the right extremity of the cross-section. The direction "left" and "right" are determined by facing in a downstream direction.

Figures 8.4 to 8.7 represent four typical cross-sections on the Salmon and North Rivers. These cross-sections were located at: 1) Salmon River at confluence with North River; 2) Salmon River estuary at Highway 102; 3) Salmon River at outlet, and 4) North River at Stewart Bridge. A complete summary of cross-sections is available as supplementary information.

Only two sections, FC 170-5 and FC 170-6, have conveyance and embayment areas. Other cross-sections of flood channels inputted in the model have conveyance areas only. Water volumes which enter the flood channels may return to the main channels by a combination of flow through the aboiteaux and over the dykes at the prescribed breach points when tide levels subside. The embayment zones for flood channel 170 (at the CNR bridge on the Salmon River) are required for stability purposes due to sharp differences in cross-sectional area between sections.

The embayment provision was also employed in the reach above the Marshall's Corner bridge on the North River. In employing this provision in the upper reaches, the main channel or flow conveyance area was assumed to be contained within the cross-section between the left and right banks at points corresponding to a "bankfull" condition. The rest of the cross-section area outside the two points for the "bankfull" condition is treated as a storage zone with no flow velocity. The model computes the same stage in the main channel and the embayment at each mesh point cross-section.

Table 8.4 provides a summary of parameter values for each of the 11 breach structures used in the Truro model. Three different types of breach structures are in operation, namely: culvert with dyke, aboiteaux with dyke, and dyke only. The table also summarizes locations with respect to the model schematic configuration together with a summary of dimensions. For the purpose of modelling three-barrel circular conduits in breach structures 4 and 6, it was necessary to calculate equivalent hydraulic characteristics by conversion to an equivalent rectangular section. This was done and input to the model with the same invert elevations and equal cross-sectional areas.

It was determined from the field surveys that the lower chord elevations of bridge openings are located above expected open water flood elevations. Therefore, it was assumed that it was not necessary to provide additional hydraulic losses specifically at bridge structures beyond accounting for the configuration of the channel and floodplain at each bridge location. (Surveyed sections at bridge openings include abutments and any piers in the sections.)

8.4 Model Calibration and Validation

To ensure reliable simulation results, the hydraulic model was calibrated by adjusting the river bed roughness coefficient (Manning's "n") at every surveyed cross-section.

It is desirable that the model calibration and validation should cover both high and low flow events at different times of the year to ensure that "n" values used in the model are representative of a wide range of dynamic and seasonal flow regimes. However, the main interest in this study was the determination of peak water levels associated with significant flood events. Therefore, recorded water level and discharge data from major flood events should be used for model calibration and validation. The data collected for this purpose were discussed in Chapter 5.0. However , neither the event of April 29, 1982 nor that of April 16, 1980 were large enough to result in flooding over the dykes and along the flood channels.

The model was calibrated using water surface elevations measured April 19, 1982 at various gauging stations along the Salmon and North Rivers (see Chapter 5.0).

An iterative procedure was used to calibrate the model by varying the Manning's "n" roughness coefficient until a satisfactory comparison of simulated and recorded levels was achieved. The roughness coefficients resulting from the model calibration are summarized in Table 8.5. It was assumed that river channel reaches 40, 50, 60, 240 and 270 have narrow ranges of "n" that vary linearly from a generally higher value to a lower value as water level rises. It was also assumed that the "n" value assigned to flood channels could vary linearly from 0.100 to 0.050 as water level rises. The higher values in the flood channels were assumed to reflect the effects of additional vegetation and other obstructions to flow. The roughness coefficients which were determined and assigned for the study area are consistent with those published by

TABLE 8.5

Manning's Roughness Coefficients

Reach	Upstream Node Water Level Range (m)	n range*	Downstream Node Water Level Range (m)	n range*
10	38.11 - 34.15	.025	34.76 - 30.79	.030
30	39.63 - 33.54	.025	37.20 - 31.10	.030
40	35.21 - 30.64	.030 - .035	30.18 - 25.61	.027 - .029
50	33.38 - 25.61	.027 - .029	18.29 - 10.52	.027 - .029
60	15.09 - 10.52	.029 - .035	12.04 - 7.47	.030
70	11.59 - 7.47	.032	11.59 - 7.47	.030
80	15.24 - 11.13	.050 - .100	12.96 - 8.84	.050 - .100
90	15.24 - 11.13	.050 - .100	11.59 - 7.47	.050 - .100
100	13.41 - 7.01	.050 - .100	13.41 - 7.01	.050 - .100
110	12.80 - 8.23	.050 - .100	11.43 - 6.86	.050 - .100
120	41.16 - 36.13	.030	32.01 - 26.98	.030
130	32.01 - 26.98	.030	26.37 - 21.34	.024
140	27.59 - 21.19	.024	16.46 - 10.06	.027
150	16.92 - 10.06	.030	13.57 - 6.71	.030
160	14.33 - 6.10	.035	13.87 - 5.64	.021 - .030
170	15.85 - 12.65	.050 - .100	12.20 - 8.99	.050 - .100
180	14.33 - 8.84	.050 - .100	14.33 - 8.84	.050 - .100
190	13.41 - 8.38	.050 - .100	13.41 - 8.38	.050 - .100
200	12.50 - 8.45	.050 - .100	12.50 - 8.45	.050 - .100
210	12.50 - 8.38	.050 - .100	12.50 - 8.38	.050 - .100
220	12.50 - 9.30	.050 - .100	11.13 - 7.93	.050 - .100
230	18.90 - 10.21	.030	18.29 - 9.60	.030 - .045
240	14.63 - 9.60	.030 - .045	10.98 - 5.95	.030 - .045
250	14.02 - 5.79	.030 - .100	13.57 - 5.34	.030 - .100
260	12.65 - 7.62	.050 - .100	11.28 - 6.25	.050 - .100
270	14.63 - 6.10	.021 - .030	11.13 - 2.44	.025 - .030

* linear variation from high level to low level

Chow (1959), Henderson (1966), and the U.S. Bureau of Reclamation (U.S. Dept. Interior 1978).

With reference to the comparison of measured and simulated stage hydrographs, it is evident that an acceptable level of accuracy has been achieved through model calibration (see Figures 8.2 and 8.3). This is better illustrated by reference to Table 8.6 which shows that the overall average percentage difference of the recorded versus computed peak water levels is less than 7% corresponding to an overall average of difference in level of only 0.14 m. The standard error of estimate was found to be about 0.17 m. It was concluded that this level of accuracy was adequate for the present investigation.

The model was subsequently validated using the water level data recorded for the event of April 16, 1980. The accuracy of the simulated water levels for the validation event is illustrated with reference to Table 8.7 which indicates that the percentage difference in peak levels is less than 2% with a corresponding standard error of level of about 0.107 m.

It was concluded that the simulation model produced acceptable simulation results. The model was then used to simulate different scenario runs required for estimating flood levels associated with design floods (see Section 8.5) and for undertaking a sensitivity analysis of the effects of variations in hydraulic factors on flood profiles along the study reach.

Data from larger flood events would be required to more accurately verify the discharge simulation over the dykes and along floodways. However, sensitivity testing has indicated that additional refinement to the model would not likely result in significant changes to design flood elevations (see Section 8.6).

TABLE 8.6
APRIL 29, 1982 EVENT AT 18:00 HOURS

Gauging Station	Recorded Level (m) A	Computed Level (m) B	Net Difference (m) [B-A]=C	Depth of Water (m) D	% Difference C/D*100 E
S2	6.71	6.71	0.00	4.36	0.0
S3	6.95	6.73	0.22	4.28	5.14
S4	6.89	7.13	0.24	1.91	12.57
S6	7.46	7.61	0.15	2.07	7.25
S7	8.90	9.00	0.10	3.20	3.13
S8	11.44	11.79	0.35	1.88	18.62
S9	16.44	16.59	0.15	1.34	11.19
S10	21.31	21.32	0.01	1.38	0.72
S11	28.67	28.89	0.22	1.79	12.29
WSC gauge*	25.89	25.98	0.09	2.57	3.50
N1	7.41	7.48	0.07	1.58	4.43
Bailey bridge	8.66	8.62	0.04	1.79	2.23
Sum			1.64		81.07

The overall average of difference in level $= \overline{C} = \Sigma\ C/N = 0.14$ m

The overall average of percentage difference $= \overline{E} = \Sigma\ E/N = 6.75\%$

The standard error of level $= \sigma = \left[\frac{\Sigma(B-A)^2}{N}\right]^{1/2} = 0.17$ m

* North River at North River Sta. 01DH004

TABLE 8.7
APRIL 16, 1980 EVENT AT 02:00 HOURS

Gauging Station	Recorded Level (m) A	Computed Level (m) B	Net Difference (m) [B-A]=C	Depth of Water (m) D	% Difference E
S3	9.30	9.31	0.01	6.69	0.15
S4	9.57	9.54	0.03	4.30	0.69
S7	10.03	9.85	0.18	4.04	4.45
Sum			0.22		5.29

The overall average of difference in level $= \bar{C} = \Sigma C/N = 0.07$ m

The overall average of percentage difference $= \bar{E} = \Sigma E/N = 1.76\%$

The standard error of level $= \sigma = \left[\frac{\Sigma(B-A)^2}{N}\right]^{1/2} = 0.11$ m

8.5 Design Flood Profiles

8.5.1 General

The main purpose of the study was to calculate the 1:20 and 1:100 year flood profiles along the study area. The design conditions for discharge were discussed in Chapter 6.0 and the design characteristics for tidal boundary conditions are presented in Chapter 7.0.

Subsequent to the model calibration and validation discussed in the previous sections, the design conditions for discharge were input to the model as upstream boundary conditions as point lateral inflows or as uniformly distributed lateral inflows and the design characteristics for tides were input as the downstream boundary conditions.

The model then simulated the effects of channel and floodplain routing automatically, taking into account dyke overtopping and other flow transfers where appropriate. A series of stage hydrographs showing the time variation in flood levels was output at several selected locations in order to analyse the routing of the flood hydrograph and to study the effect of other factors on flood peaks. The peak stage at each location was then extracted in order to plot the corresponding 1:20 and 1:100 year flood profiles along the study area (see Section 8.5.2). The hydraulic characteristics in the study area were then analysed as discussed in Section 8.5.3.

In order to confirm the selection of model parameters and design conditions, sensitivity analyses were then undertaken as discussed in Section 8.6. This involved re-running the model for a number of scenarios reflecting variations in basic input parameters and boundary conditions. For example, variations in tidal characteristics, discharge, and the effect of ice jams on water levels were examined in this manner, in addition to a number of other parameters (see Section 8.6).

Finally, a summary of conclusions regarding the results of the hydrodynamic modelling is presented in Section 8.7.

8.5.2 Design Flood Events

The selected design flood events are identified as follows:

1. 1:20 year discharge; 1:20 year tide (Run A)
2. 1:100 year discharge; 1:100 year tide (Run D)

The letters A and D identify the simulation according to the scenarios discussed in the section on sensitivity testing (see Section 8.6 and Table 8.8).

The corresponding 1:20 and 1:100 year flood profiles along the study reach are depicted on Figures 8.8 to 8.11. The flood profiles are summarized numerically in Table 8.9a and b. These flood levels were subsequently utilized in the plotting of floodlines on the topographic mapping of the study area.

8.5.3 Discussion of Modelling Results

Flood conditions associated with events such as the 1:20 and 1:100 year events result in significant flow depths and velocities along the study area. The upper reaches of the Salmon and North Rivers are fairly steep and are free from backwater effects from downstream due to incoming tides. For a 1:20 year flood peak levels reach 23 m at Murray Siding on the Salmon River and 27.34 m at Stewart Bridge on the North River (Table 8.8). The maximum flow depths and velocities are summarized for selected locations in Table 8.10. A variation in discharge velocities can occur along the system, ranging from over 3.0 m/s near the confluence of the

TABLE 8.8

Summary of Sensitivity Testing/Simulation Results

Location		Simulation Results - Summary															
* Scenario Identification		A		B		C		D		E		F		G		H	
Description/ Location No.	Sect. No.	Peak Level (m)	Dis-charge (m^3/s)	Peak Level (m)	Dis-charge (m^3/s)	Peak Level (m)	Dis-charge (m^3/s)	Peak Level (m)	Dis-charge (m^3/s)	Peak Level (m)	Dis-charge (m^3/s)	Peak Level (m)	Dis-charge (m^3/s)	Peak Level (m)	Dis-charge (m^3/s)	Peak Level (m)	Dis-charge (m^3/s)
Salmon River																	
1) At outlet	R-270	10.10	1267.0	8.50	841.0	10.10	1296.7	10.40	1448.9	8.50	1012.8	10.40	1675.6	6.12	362.8	6.31	152.4
2) At Board Landing Br.	R-270	10.12	542.0	8.70	523.8	10.11	647.2	10.34	633.5	8.76	598.0	10.41	852.3	7.36	334.8	8.89	132.6
3) At confluence with North River	R-270	10.11	359.7	9.28	348.4	10.18	452.8	10.30	434.5	9.61	421.5	10.53	607.4	9.27	316.0	10.10	125.5
4) At Park St. Bridge	R-160	10.38	205.7	10.05	206.5	10.72	275.1	10.66	247.4	10.46	250.3	11.13	422.3	10.07	184.8	10.54	167.4
5) At CNR Bridge	R-150	13.30	246.9	13.30	247.0	13.88	384.1	13.77	334.1	13.77	334.1	14.32	571.7	12.46	228.4	13.74	230.3
6) At Murray Siding	R-140	23.00	244.4	23.00	244.4	23.79	402.1	23.51	338.5	23.51	338.5	24.53	592.5	22.86	222.1	22.86	222.1
7) At Valley	R-120	39.23	212.8	39.23	212.8	40.05	361.7	39.71	294.9	39.71	294.9	40.78	536.3	39.05	187.5	39.05	187.5
North River																	
8) Onslow Road Bridge at Marshall's Corner	R-70	9.99	82.1	9.45	80.8	10.03	77.0	10.11	83.9	9.63	78.5	10.18	83.5	9.45	82.6	9.97	44.2
9) At TransCanada Hwy.	R-60	11.89	160.3	11.88	160.3	12.58	279.4	12.20	219.7	12.25	219.8	13.00	406.9	11.76	140.6	11.76	140.6
10) At Stewart Bridge	R-50	27.34	145.9	27.34	145.9	28.18	261.8	27.80	200.5	27.80	200.4	28.81	381.2	27.15	125.1	27.15	125.1
11) At Confluence with West North River	R-40	32.19	141.1	32.19	141.1	32.77	256.3	32.47	193.9	32.47	193.9	33.27	375.6	32.06	119.9	32.06	119.9
McClures Brook																	
12) D/S of Hwy 102	R-250	9.29	33.4	8.99	34.1	9.68	43.3	10.08	44.8	9.28	45.8	10.44	58.8	8.91	36.0	9.95	31.4
13) U/S of Hwy 102	R-240	9.30	36.6	9.04	37.2	9.68	47.8	10.02	48.9	9.31	49.2	10.43	67.1	8.97	37.5	9.95	28.4
14) U/S of D.A.R.	R-230	11.45	50.0	11.45	52.0	11.55	63.5	11.57	64.8	11.56	64.7	12.00	96.0	12.82	51.3	12.81	51.0
Floodplain S. of Salmon River																	
15) U/S of Hwy 102 Viaduct	R-210	9.36	34.5	8.98	9.7	9.68	24.3	10.09	147.6	9.28	10.8	10.43	137.7	8.89	5.2	9.96	41.6
16) South Flood Channel at Park St.	R-190	9.28	0.0	8.98	0.0	9.68	15.4	10.07	11.8	9.28	0.0	10.51	86.7	8.89	0.0	9.96	4.5
17) South of North R. Confluence	R-200	9.28	41.2	8.98	2.0	9.68	51.1	10.06	71.9	9.28	2.0	10.49	75.8	8.89	0.4	9.96	74.7
Floodplain N. of Salmon River																	
18) N. of McCurdy Bk. Aboiteau	R-110	9.45	34.2	9.22	33.3	9.88	39.6	9.93	41.6	9.67	43.2	10.88	84.5	9.26	30.3	9.93	25.4
19) N. side of Farnham Bk. Aboiteau	R-100	10.20	34.4	9.81	23.0	10.45	32.7	10.44	30.8	10.15	29.7	10.77	36.8	9.91	24.4	10.43	22.6
20) Between Farnham Bk & Park St.	R-180	10.24	59.1	9.87	44.3	10.51	100.8	10.49	88.6	10.23	80.5	10.82	137.8	9.96	43.9	10.46	61.5
21) Between Park St. and CNR	R-170	10.38	4.3	10.06	2.4	11.12	12.8	10.65	6.1	10.50	4.8	11.95	36.4	10.08	2.4	10.54	4.0
22) McCurdy Brook north of CNR	R-80	9.46	2.0	9.26	2.2	9.84	2.0	9.87	3.6	9.67	4.2	10.29	2.5	9.47	2.4	9.89	2.1

* Scenario Identification :

A - 1:20 year flow, 1:20 year tide
B - 1:20 year flow, typical spring tide
C - 1:20 year flow (95% CL), 1:20 year tide
D - 1:100 year flow, 1:100 year tide
E - 1:100 year flow, typical spring tide
F - 1:100 year flow (95% CL), 1:100 year tide
G - 1971 event, open water flood conditions (no tide)
H - 1971 event, ice jam (no tide)

TABLE 8.9a
1:20 YEAR FLOOD PROFILES *

Reach No.	Flood Elevations at Odd-numbered mesh Points (m)																				
	(1)	(3)	(5)	(7)	(9)	(11)	(13)	(15)	(17)	(19)	(21)	(23)	(25)	(27)	(29)	(31)	(33)	(35)	(37)	(39)	(41)
10	35.98	35.02	34.13	33.25	32.18																
30	35.34	34.00	33.28	32.18																	
40	32.18	30.73	29.76	28.91	27.97	27.34															
50	27.34	26.10	25.22	24.28	22.97	21.70	20.57	19.22	17.62	16.04	14.52	13.14	11.87								
60	11.87	11.10	10.55	9.77																	
70	9.77	9.86	9.99	10.07	10.11	10.12	10.12														
80	9.77	9.58	9.46	9.46																	
90	9.90	9.90	10.10	10.20																	
100	10.20	10.20	10.20																		
110	9.44	9.44	9.44	9.44	9.44																
120	39.[illegible]2	38.19	36.80	35.17	33.76	32.39	30.92	29.17	28.77												
130	28.77	28.36	27.56	26.34	25.12	23.94	22.99														
140	22.99	22.09	21.24	20.29	19.36	18.47	17.58	16.61	15.83	15.33	14.62	13.81	13.38	13.30							
150	13.30	12.48	11.69	11.23	10.80	10.53	10.40														
160	10.38	10.27	10.21	10.13	10.12																
170	-	-	10.40	10.40	10.40																
180	10.32	10.24	10.22	10.22	10.20																
190	9.28	9.28	9.28	9.28	9.28																
200	9.37	9.28	9.28																		
210	9.28	9.28	9.28	9.36																	
220	9.36	9.30	9.30																		
230	12.37	11.45	11.26																		
240	11.26	10.16	9.35	9.30																	
250	9.30	9.29	9.29																		
260	9.36	9.30	9.29																		
270	10.12	10.12	10.12	10.11	10.11	10.11	10.10	10.10	10.10	10.11	10.12	10.12	10.13	10.12	10.12	10.11	10.10	10.10	10.10	10.10	10.10

* Run A of Table 8.8
Refer to Table 8.3 for mesh point identification

TABLE 8.9b

1:100 YEAR FLOOD PROFILES *

Reach No.	Flood Elevations at Odd-numbered Mesh Points (m)																				
	(1)	(3)	(5)	(7)	(9)	(11)	(13)	(15)	(17)	(19)	(21)	(23)	(25)	(27)	(29)	(31)	(33)	(35)	(37)	(39)	(41)
10	36.25	35.29	34.41	33.46	32.46																
30	35.55	34.18	33.44	32.46																	
40	32.46	31.07	30.13	29.26	28.27	27.79															
50	27.79	26.47	25.66	24.76	23.32	21.99	20.82	19.47	17.89	16.36	14.87	13.46	12.18								
60	12.18	11.45	10.84	9.80																	
70	9.80	9.92	10.11	10.23	10.29	10.30	10.31														
80	9.80	9.74	9.87	9.87																	
90	9.[illegible]	9.95	10.26	10.42																	
100	10.44	10.44	10.44																		
110	9.93	9.93	9.93	9.93	9.93																
120	39.70	38.63	37.16	35.70	34.29	32.86	31.36	29.49	29.22												
130	29.22	28.82	28.03	26.81	25.60	24.44	23.50														
140	23.50	22.48	21.59	20.65	19.73	18.86	17.98	16.99	16.21	15.71	15.01	14.13	13.83	13.77							
150	13.77	13.44	12.15	11.63	11.18	10.85	10.68														
160	10.66	10.53	10.46	10.29	10.31																
170	13.47	12.34	10.68	10.68	10.68																
180	10.58	10.49	10.47	10.46	10.44																
190	10.08	10.07	10.07	10.06	10.06																
200	10.06	10.06	10.06																		
210	10.06	10.04	10.06	10.09																	
220	10.09	10.02	10.02																		
230	12.44	11.56	11.66																		
240	11.66	10.46	10.08	10.02																	
250	10.02	10.08	10.04																		
260	10.09	10.05	10.04																		
270	10.31	10.30	10.29	10.29	10.28	10.28	10.27	10.27	10.30	10.31	10.33	10.33	10.35	10.36	10.37	10.38	10.38	10.39	10.39	10.39	10.40

* Run D of Table 8.8
Refer to Table 8.3 for mesh point identification
An overall adjustment of 0.5 m required for all values for floodline plotting.

TABLE 8.10

Maximum Flood Depths and Velocities

	Run A		Run D	
	Flow Depth (m)	Velocity (m/s)	Flow Depth (m)	Velocity (m/s)
R40 - 0 m - Confluence with West North River	1.79	2.57	2.08	2.89
R60 - 0 m T/C Hwy. Bridge	1.69	2.34	2.00	2.65
R70 - 0 m North River	2.66	1.20	2.69	1.29
R130 - 0 m below Valley Bridge	2.16	1.30	2.61	1.39
R150 - 0 m Walker Street Bridge	4.55	1.39	5.02	1.56
*R240 - 2204 m Upstream of Hwy. 102	3.66	0.69	4.09	0.82
*R270 - 0 m Salmon/North confluence	4.58	3.13	4.76	3.87

* backwater effects are present at these locations

During the occurrence of events such as the 1:100 year flood, it was found that discharge rates in the various flood channels can be high, but because of the large cross-sectional areas serving as flood storages, velocities are usually very low in comparison with that in the main channels.

Downstream from the CNR bridge on the Salmon River and the Highway 104 Bridge on the North River, the effects of tidal influences become evident. At the confluence of the Salmon and North Rivers, peak levels reach 10.11 m for a 1:20 year flood, whereas the peak level would be about 0.7 m lower should the tides not be present. Further downstream from the confluence, the tidal effects can be so dominant that the flood peaks become insignificant in comparison.

Due to the complex nature of flows in the flood channels and the occurrence of reversed flows caused by high tides, parallel comparison of the flow regime in the flood channels with that in the main channel is very difficult. Nevertheless, the storage effects of the flood channels are very clearly illustrated by the stage hydrographs determined for all flood channels and plotted at selected locations as shown on Figures 8.12 and 8.13. The peak stages on the flood channel usually occur later than the flood peaks on the main channel and are maintained for a longer period of time. It was also found that for flood stages in the Truro Dykeland Marsh area the peak stage caused by the second tide peak (the highest) is surpassed by the one caused by the third tide peak due to insufficient time for draining the freshwater flood inflows from this area.

The simulated peak levels for flood channels 190, 200, 210 and 220 were found to be almost identical. The variations in flood stage in the flat flood channels south of the Salmon River in response to the combined flood/tide event is different from that in the main channels. Only one flat peak starts to develop after the flood channels (Reaches 190, 200, 210, 220 and 260) receive significant amounts of flow over the dykes, which occurs when the tide peak and the flood peak meet. The stage hydrograph then recedes very slowly mainly by discharging through the aboiteaux west of the Highway 102 viaduct (Breach 6).

On the north side of the Salmon River, overflow to flood channel R170 was found to be quite small (less than 8 m^3/s for a short duration). On the other hand, flood channel R180 was found to receive a substantial amount of overflow during the 1:100 year event via breach structure No. 7, resulting in a flood level nearly equal to that in the main channel (channel R160).

The peak 1:100 year level at the lower end of flood channel R100 (10.44 m) was found to be determined by the peak water level at breach structure No. 3. On the other hand, the peak water level at the upper end of flood channel R90 (9.95 m) was found to be controlled by water levels in the North River just above Onslow Road Bridge at Marshall's Corner. The peak water levels are related to the occurrence of the arrival of the peak of the flood wave rather than any significant backwater effect from the bridge.

The peak 1:100 year levels in flood channel R110 (9.93 m) are a consequence of high flow over the dyke at breach structure No. 4. The storage area is large and flood levels recede slowly by discharge into the main channel.

8.6 Sensitivity Analyses

8.6.1 General

The relative effect on flood profiles of various input parameters and model coefficients was examined by means of sensitivity analyses. This was done by varying a selected parameter and comparing the simulation results to the basic 1:20 and 1:100 year flood profiles. The following simulations were undertaken:

A - 1:20 year flow, 1:20 year tide
B - 1:20 year flow, typical spring tide

C - 1:20 year flow (95% C.L.), 1:20 year tide
D - 1:100 year flow, 1:100 year tide
E - 1:100 year flow, typical spring tide
F - 1:100 year flow (95% C.L.), 1:100 year tide
G - 1971 Event; Open water; No tide
H - 1971 Event; Ice Jam; No tide

The 1:20 and 1:100 year flood profiles previously discussed are identified as runs A and D in the above list. Runs A and D are subsequently discussed as base conditions for comparative purposes.

The following parameters and characteristics were examined by means of analysing and comparing results from the above simulations:

1. discharge variations
2. tidal variations
3. timing of tides, discharge, lateral inflows
4. Manning's "n"
5. bed elevation
6. dyke elevation
7. crest length
8. ice conditions

Stage and discharge hydrographs were output from the model at a number of locations along the study area for each simulation scenario. Flood profiles were also computer-plotted for each scenario. The results were then analysed and compared to assess the relative sensitivity of each of the above parameters on the flood levels. The resulting comparisons of water levels for the various simulation scenarios are summarized in Table 8.8 which is referred to extensively in the following discussion of parameter sensitivity.

8.6.2 Discharge Variations

The upper 95% confidence limit flows were input (Runs C and F; see Table 8.8) to the hydrodynamic model in order to assess the impact of variations in discharge on peak flood levels. It was found that along the steeper reaches in the study area, which generally have narrower flow conveyance areas compared to the downstream estuary portion, that the effects of discharge variations are more significant than in the estuary where these effects are almost negligible. For example, comparing Runs A and C, the 1:20 year peak flows at the +95% confidence level are 70-80% higher than the mean 1:20 year peak flows. This resulted in higher overall peak levels with the most significant difference in peak level, 0.79 m, occurring at Murray Siding on the Salmon River. Since the tide was identical for these 2 scenarios, the corresponding increase in peak level at the confluence of the North and Salmon Rivers was only 0.07 m. The reduced impact of peak flows on flood levels in the estuary is attributed to the effect of tides and the overtopping of dykes and resulting storage of flood waters along the flood channels. For example, peak levels along the flood channels are about 0.4 m higher, on average, comparing results of scenario C to scenario A.

The results of changes in discharge for the 1:100 year event are evident by comparison of Scenarios D and F (Table 8.8). The 1:100 year peak flows at the +95% confidence level for the input hydrographs are 82-94% higher than the mean 1:100 year peak flows. Higher overall peak water levels, up to 1.3 m in the flood channel north of the Salmon River, between Park Street and the CNR bridge (R170), result from the higher flood peak. At high flood stages in the main channels, attenuation effects of the flood peak by overtopping into the flood channels become more pronounced. The corresponding increase in peak levels in flood channels, e.g. 0.34 m in R210, becomes more significant than for the 1:20 year flood event.

8.6.3 Tidal Variations

Tidal variations influence flood levels up to the CNR bridge on the Salmon River and the Highway 104 bridge on the North River. For example, comparing Runs A and B, as summarized in Table 8.8, an increase of 1.60 m in tide peak resulted in an increase of 0.83 m in simulated peak level at the confluence of the Salmon and North Rivers and 0.34 m at the Park Street Bridge. Also, an increase of 1.90 m in tide peak resulted in an increase of 0.69 m in peak level at the confluence and 0.20 m at the Park Street Bridge when comparing Run D and Run E (i.e. replacement of the 1:100 year tide with a typical spring tide).

In the case of 1:100 year flows, the effects of replacing the 1:100 year tide with typical spring tide on peak levels and flows are mainly felt in the flood channels and the Salmon estuary. The peak flood level dropped by about 0.8 m for flood channels on the south side of the Salmon River and by about 0.3 m on the north side. Therefore, the peak water levels in the Salmon River estuary are particularly sensitive to tidal variations.

8.6.4 Timing of Tides, Discharges and Lateral Inflows

It was found that any timing changes in lateral inflows would not change the peak water levels. This is attributed to the relatively small magnitude of lateral inflows.

A number of additional simulations were undertaken for the 1:20 year event by varying the start of rise of the freshwater flood hydrograph relative to the time of peak tidal conditions. It was found that if the tidal peak was lagged by 2.6 hours behind the flood peak (at the upstream limit of the study area) that the highest peak stage occurs in the area of the Park Street Bridge.

It was found that, with the same magnitude of tides and floods, any lagging of the tide peak shorter than 2.6 hours results in slightly higher peak levels at the CNR bridge on the Salmon River and slightly lower peak level at the Park Street Bridge on the Salmon River. This is attributed to advancement of tide water upstream on the Salmon River. On the other hand, any lagging of the tide peak longer than 2.6 hours would result in higher peak level at the confluence of the Salmon River and North Rivers with lower peak level at the Park Street Bridge. This is attributed to the advancement of the flood peak further downstream on the Salmon River to a point where it meets the incoming tide peak downstream of the Park Street Bridge. It was also found that the time of travel in the flood peak to arrive at the Park Street Bridge from the upstream end of the study area is about 3.0 hours.

A 2.6 hour lag between the tide and incoming flood hydrograph was, therefore, adopted since flood levels in the Park Street area were viewed to be more critical in the present investigation.

8.6.5 Manning's "n"

The Manning's roughness coefficients were derived during the model calibration and validation. Variations in Manning's "n" were made for selected locations for the 1:20 and 1:100 year flood profiles (Runs A and D).

In the case of Run A, when Manning's "n" for the lower Salmon estuary (R270) was increased by 50%, the increase in peak water level was less than 0.1 m. The relatively small change is attributed to the control of peak levels by tidal conditions and it is concluded that the tidal flow through the estuary channel is not sensitive to changes in the roughness coefficient.

Furthermore, in the case of Run D, when Manning's "n" for R160 was increased from 0.035 to 0.055, the increase in peak level was found to be 0.09 m at the CNR bridge on the Salmon River and 0.22 m at the Park Street Bridge. Since the model has been calibrated and the expected variations would be less than those assumed in the sensitivity testing, it is concluded that flood levels in the study area are relatively insensitive to changes in Manning's "n".

8.6.6 Bed Elevation

Erosion and sedimentation can cause significant changes in channel bottom elevations in the tidal zone of the Salmon River (see Section 3.0). Therefore, a sensitivity simulation for Run A was undertaken to examine the effects on flood levels of a possible bed elevation change at the confluence of the Salmon and North Rivers.

At the confluence, it was found that the increase in bed elevation by 0.6 m could result in an increase in water level of up to 0.4 m at low flow depths, whereas at the greater flow depths associated with flood peaks, the water level was insensitive to bed elevation changes. It was also found that such a change gave no significant change in levels upstream at the CNR bridge for both low flow and peak flow conditions. However, the impact of changes in bed level on ice jam flooding could be more significant.

8.6.7 Dyke Elevation

The existing dyke crest elevation of 9.6 m was used to represent the north side of the Salmon River at the Park Street bridge (breach structure 7 at node 26). The 1:20 year discharge and tide were utilized to assess the effects of raising the top of dyke by 0.9 m on upstream and downstream water levels. It was found that upstream water levels were not affected appreciably by the higher dyke crest. However, water levels just downstream were slightly higher due to the fact that flows over the dyke to the flood channel (reach 180) did not occur until levels reached 10.5 m at node 26. However, the peak levels remained unchanged in both cases.

The area represented by the flood channel R210 east of the Highway 102 viaduct in the model, is one of the most flood prone areas.

The simulated flood peak in this reach was found to be directly affected by the dyke crest elevations on the south side of the Salmon River, particularly those represented by breach structure Nos. 9 and 11. In the case of Run D, when the dyke elevation at breach structure No. 9 was raised from 9.85 m to 10.35 m, the flood peak along flood channel R210 remained unaffected. The corresponding flood level at the confluence with the North River changed from 10.31 m to 10.36 m.

When the dyke elevation at breach 9 remained unchanged, the elevation at breach 11 was raised from 9.91 m to 10.41 m, the resulting flood peak at the confluence was raised from 10.31 m to 10.41 m due to reduced flow relief through breach 11. Consequently, the resulting flood peak in reach R210 lowered from 10.09 m to 9.71 m.

Peak flood elevations in flood channel R210 were found to be sensitive to the dyke elevation at breach 11 from which reach R210 received the bulk of the overflow. However, along the two main channels, simulated flood peak levels did not change appreciably with higher dyke crests. This is attributed to the fact that the flow rates over the dyke were relatively low when compared with the peak flood discharge rates in the main channel.

8.6.8 Crest Lengths

Dyke crest lengths adopted in the model were obtained from available contour maps and local information. These overflow lengths were difficult to assess from the available information and as a consequence sometimes required adjustment in order to avoid excessive instability of flow over the dyke crests.

The 1:100 year flood (Run D) was simulated assuming an increase in the overflow crest at breach No. 9 from 227 m to 305 m and at breach 11 from 1524 m to 2744 m. The corresponding increase in flood level in channel R210 was 0.13 m. Changes in flood level at other locations were all less than 0.13 m. Similar analyses for the 1:20 year flood resulted in even smaller differences in levels due to much lower volumes of flow over the dykes.

Therefore, it was concluded that peak water levels are not sensitive to the crest length for overflow at the breach structures, provided the initial values are realistic.

8.6.9 Ice Conditions

It is well known that the effect of ice accumulations can cause significant flood levels in the study area (see Sections 3.0 and 4.0). Since only very limited ice data and information on flooding caused by ice jams were available from historical records, a full-fledged series of simulations was considered impossible. However, the sensitivity of flood levels for assumed conditions of ice accumulation were undertaken for the flood event of February 14, 1971, for which it was known that an ice jam contributed to observed flood levels. (Aerial photographs were available which documented the extent of ice cover.) Two simulations were undertaken and are summarized in Table 8.8 as Runs G and H. Scenario G is the simulation of the February 14,1971 event without any ice or tide, and Scenario H is that of the same event with assumed ice conditions incorporated.

Physical conditions at the breach structures which existed at the time of the event in 1971 were also input as slight modifications to the model.

The February 14, 1971 flood has a magnitude, in terms of peak discharge and volume, similar to that of the 1:20 year flood, as discussed in Chapter 6.0. Ten-day hydrographs were developed from daily mean flow records of the Salmon River at Valley Bridge, and then were used as boundary condition hydrographs for Scenarios G and H. The other tributary hydrographs and lateral inflows were the same as those used in Run A, i.e. 1:20 year flood, except that no tide was input to simulate the effects of ice accumulation in the estuary. A comparison of peak levels from Run G with those of Run A indicated that the removal of tide (with no ice upstream) generally resulted in lower flood levels.

A comparison of simulated water levels (Runs G and H) to those recorded upstream of the Highway 102 viaduct and in the Walker Street bridge area is given as follows:

	Scenario G	Scenario H	Recorded
Upstream of Hwy. 102 Viaduct	8.89 m	9.96 m	10.10 m
Walker Street Bridge area	12.47 m	12.49 m	11.70 m

The simulated level (Run H) is within 0.14 m of the recorded level upstream of the Hwy. 102 viaduct, and 1.07 m higher than the simulated level assuming no ice in the reach.

On the other hand, the simulated level in the Walker Street area was 0.79 m higher than the observed level. A number of factors could contribute to this discrepancy, including; the shape of the inflow hydrograph (a flatter hydrograph would result in lower simulated levels; hourly discharge data was not available to improve the accuracy here); upstream ice jams could have affected peak flows; changes in channel characteristics, etc. since 1971; and finally, the degree of accuracy of the "observed" water level is unknown.

Nevertheless, the results of the sensitivity analyses on ice jam conditions have demonstrated that, compared to open water levels, ice jams can significantly increase peak flood levels in the study area.

Subsequent simulations assuming a 100% ice cover on the entire network resulted in significant peak level increases of 1.81 m at the

CNR bridge, 0.65 m at the confluence, 0.33 m at the North River confluence with the West North River, and 0.52 m at the Highway 104 Bridge on the North River.

8.7 Conclusions

Combining the effects of celestial tides and freshwater flood hydrographs has proven to be a challenging task. The hydraulic computations in the study area are complex due to the need to simulate overtopping of dykes, discharges through aboiteaux and flow in flood channels while the limited availability of historical data for past floods yielded only a partial calibration and validation of the 1-D model. The simulation results have proven that this technique is a versatile tool which has produced reliable results which are acceptable for the purposes of this investigation.

The sensitivity analyses assessed the relative importance of variations in several key input factors affecting simulation of peak levels. It was found that variations in flood discharge had the greatest effect on the calculated water levels, especially along the upper main channel and flood channels. The effect of discharge variations tended to reduce gradually as the flood wave proceeded downstream. This was mainly because tidal effects on the peak flood level were the greatest in the downstream reaches. The impact of tidal variations on flood levels was found to be significant in the estuary, becoming negligible at the CNR bridge on the Salmon River and the Highway 104 Bridge on the North River.

It was also found that ice jams have a significant effect on flood levels in the study area. Future ice jam floods in the study area may be simulated in more detail with the 1-D model if additional data regarding the characteristics of ice jams becomes available.

The effects of flood channel conveyance and storage capabilities were found to be very significant during high water periods. These factors contribute to the attenuation of flood peaks by creating temporary storage of part of the flood volume. Therefore, it was important to accurately incorporate the flood channels in the 1-D model of the Truro area.

In addition to the significant effects of discharge variations, tidal fluctuations and ice jam characteristics, a number of other factors were found to influence flood levels to a lesser degree. These factors include erosion and sedimentation of the channel bed, dyke elevation and crest length available for overtopping, the Manning's roughness coefficient and the relative timing of the peaks of tides and freshwater flows. Sufficient data are not presently available to fully analyse the joint probability of occurrence of all these factors, although their relative importance was addressed by means of sensitivity analyses.

The conveyance area required by the 1:20 year flood is adequate to convey the bulk of flood waters for most flood events. Therefore, the 1:20 year flood profile, as calculated by the 1-D model, should be used to determine the areal extent of the floodway within the flood risk area.

An examination of the simulation results and sensitivity analyses for the 1:100 year event has revealed that the associated flood levels are slightly below the maximum historical observed levels (see Section 3.0). A comparison of observed and simulated levels is summarized in Table 8.11. For several historical floods the water levels were significantly affected by ice jams in addition to other factors. The 1:100 year levels determined in a previous investigation incorporating the effects of ice jams in the study area are also summarized in Table 8.11 for comparison purposes. Since the present data base is inadequate to accurately simulate all factors, the adoption of an adjustment factor in association with the 1:100 year flood profile was considered to be appropriate. An overall adjustment of +0.5 m to the base 1:100 year

TABLE 8.11

Comparison of Simulated and Historical Estimates of Flood Levels

Location (1)	Maximum (2) Historical Elevation (m)	Previous (3) 1:100 Year Elevation (m)	Simulated (4) 1:100 Year Elevation (m)	Simulated 1:100 Year Plus 0.5 m (m)
a) Salmon River upstream of Highway 102 viaduct	10.1	11.0	10.09	10.59
b) Salmon River at Park Street	10.96	11.0	10.65	11.15
c) Salmon River at CNR Bridge	16.0	14.0	13.77	14.27
d) Salmon River at Montgomery Dr.	24.0 ±	N/A	23.51	24.01
e) North River at CNR Railroad Bridge	10.8	N/A	9.95	10.45

(1) Comparison at selected locations

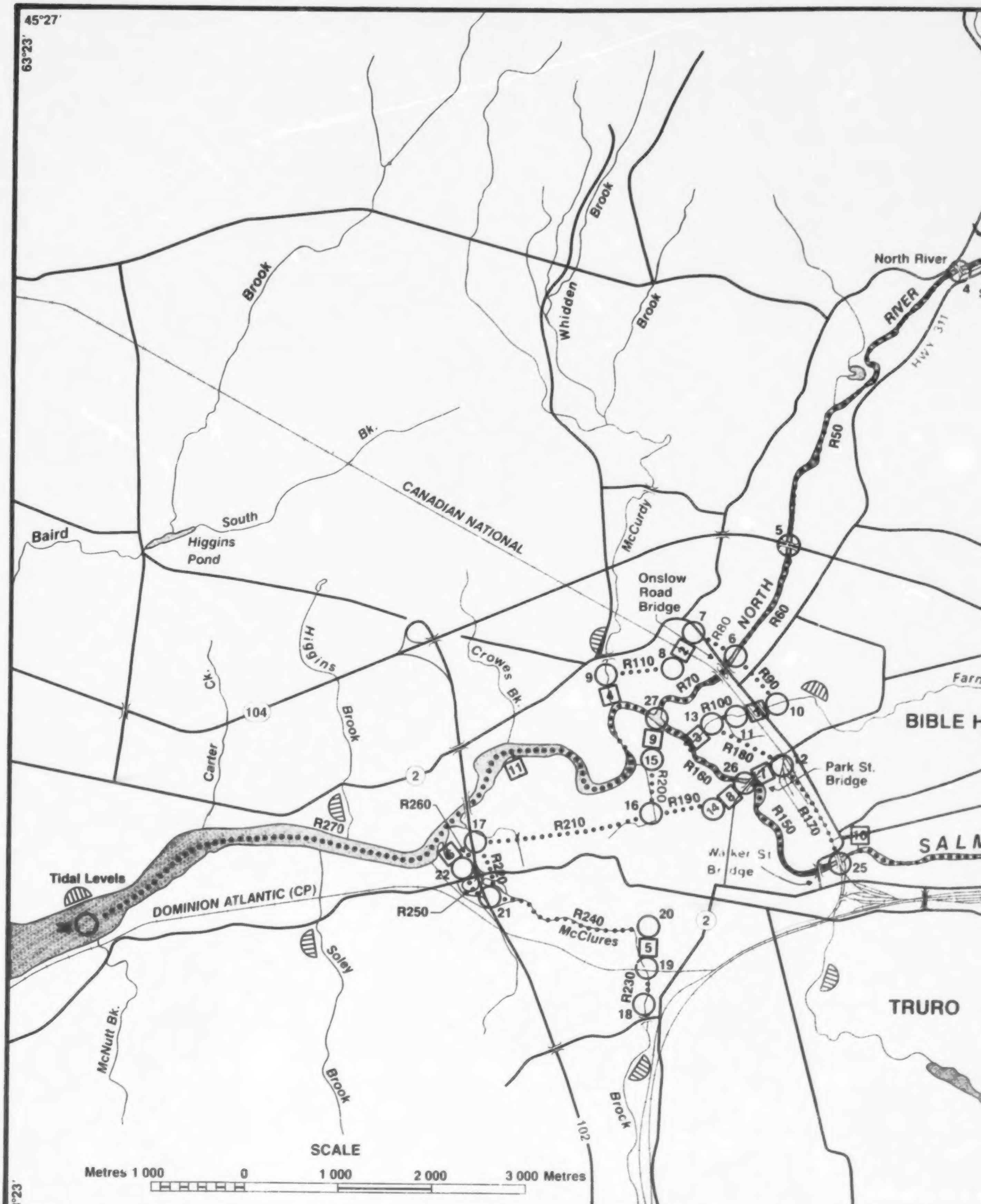
(2) Estimated Historical Elevation (see Table 3.2)
(Will include effects of ice, etc.)

(3) 1:100 Year Flood Level Estimate by Carson, 1978 (see Figure 4.1)

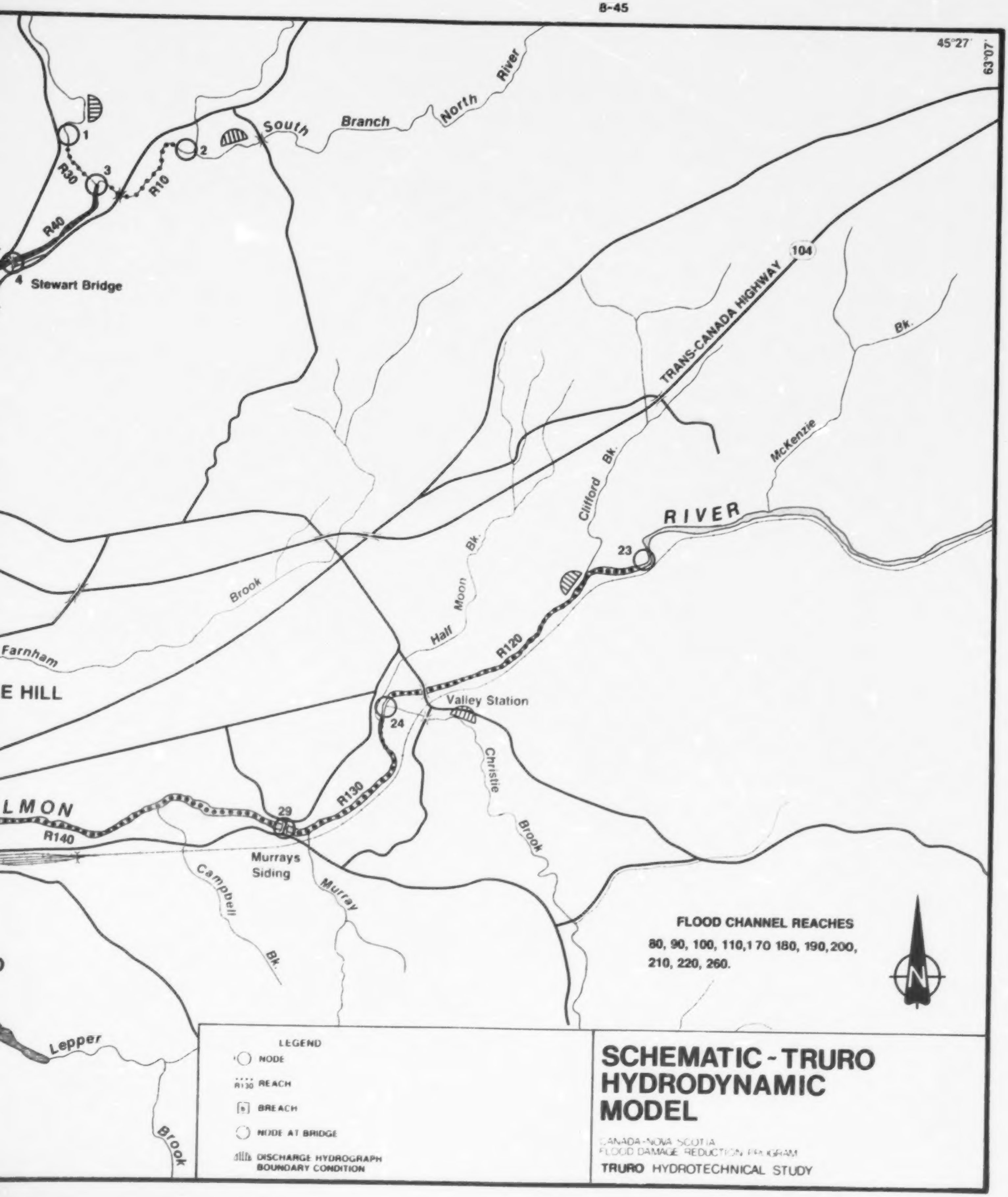
(4) From Table 8.9b

profile was considered most appropriate to account for the multitude of other factors affecting peak flood stage in the study area. The addition of 0.5 m to the computed 1:100 year flood profile and not the 1:20 year floodline would not prohibit additional development. On the other hand, future flood risk would be appropriately reduced, taking into account the many factors affecting flood levels in the Truro area, by requiring a moderate increase in the level to which structures outside the floodway should be floodproofed.

The numerical representation of the flood profiles shown on the new topographic mapping are summarized in Table 8.9.



45°27'
63°23'
Brook
Whidden
Brook
Brook
North River
RIVER
HWY 311
R50
Bk.
CANADIAN NATIONAL
South
Higgins
Pond
Baird
McCurdy
Onslow
Road
Bridge
NORTH
R60
5
4
7
R80
6
R90
8
R110
9
R70
27
R100
13
10
11
R180
15
R200
R160
26
2
Park St.
Bridge
R150
R170
16
R190
14
Higgins
Brook
Crowes Bk.
Carter
Ck.
104
2
R260
R270
17
R210
R220
22
21
R250
R240
McClures
20
19
R230
18
Tidal Levels
DOMINION ATLANTIC (CP)
Walker St
Bridge
25
SALM
Farm
BIBLE H
TRURO
Soley
Brook
McNutt Bk.
Brock
102
SCALE
Metres 1 000
0
1 000
2 000
3 000 Metres
63°23'
45°20'

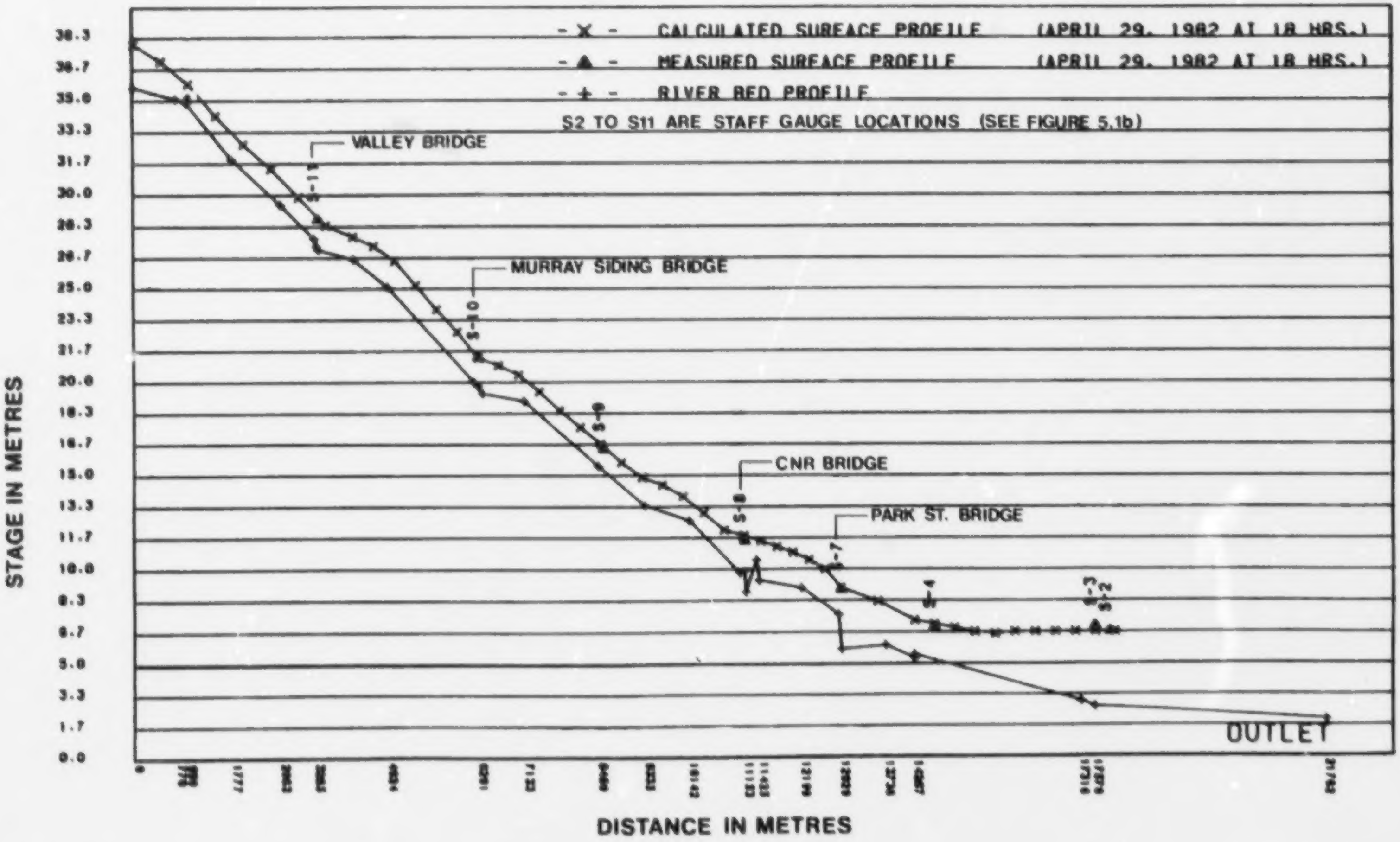

45°27'
63°07'
South Branch North River
1
2
3
R30
R10
R40
4 Stewart Bridge
104
TRANS-CANADA HIGHWAY
Bk.
McKenzie
Clifford Bk.
RIVER
23
Half Moon Bk.
Brook
Farnham
E HILL
R120
Valley Station
24
Christie Brook
LMON
R140
29
R130
Murrays Siding
Campbell Bk.
Murray
Lepper
Brook
FLOOD CHANNEL REACHES
80, 90, 100, 110,170 180, 190,200, 210, 220, 260.
N
LEGEND
NODE
R130 REACH
BREACH
NODE AT BRIDGE
DISCHARGE HYDROGRAPH BOUNDARY CONDITION

SCHEMATIC - TRURO HYDRODYNAMIC MODEL

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

TRURO HYDROTECHNICAL STUDY

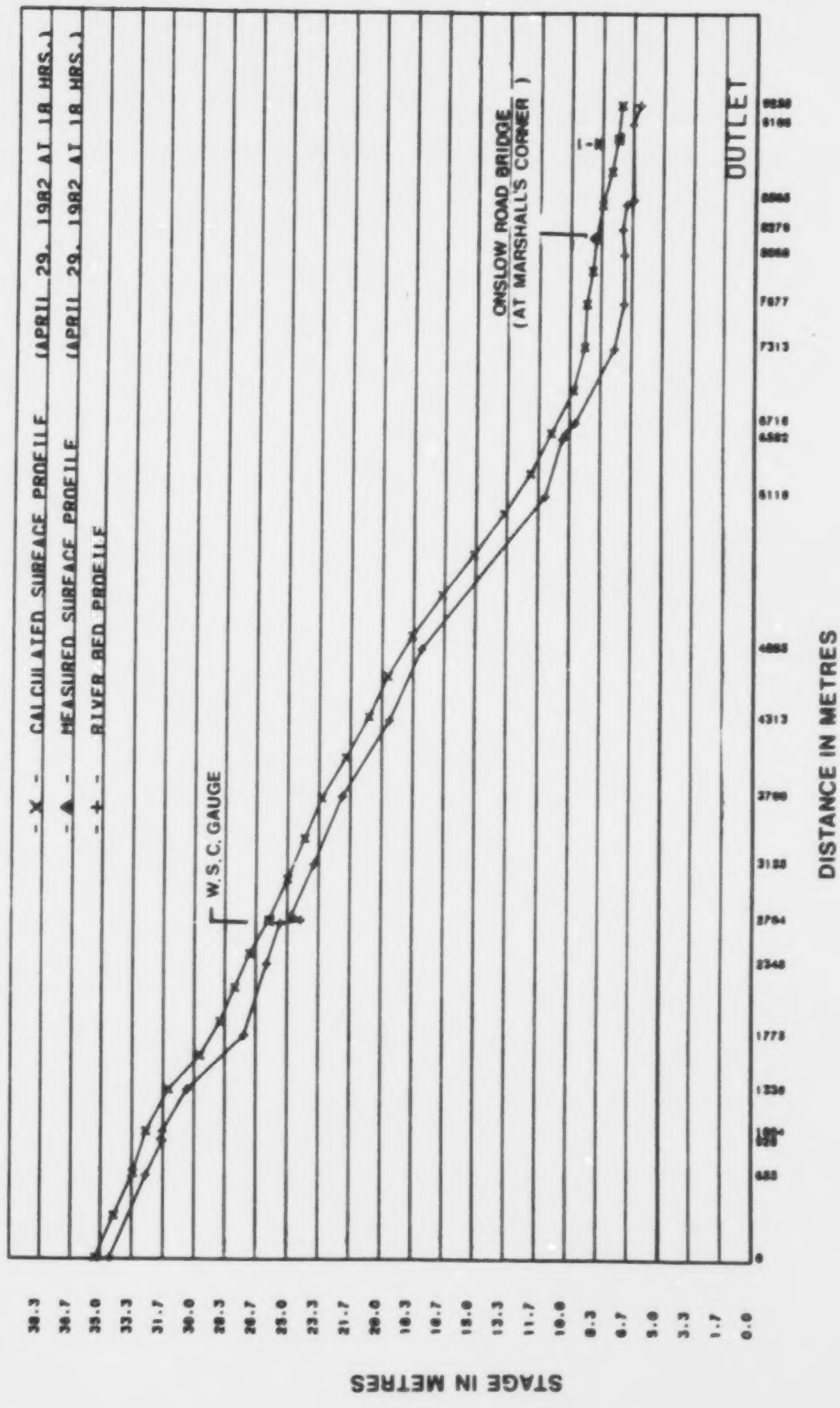
FIGURE 8.1



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

SALMON RIVER PROFILES
APRIL 29, 1982

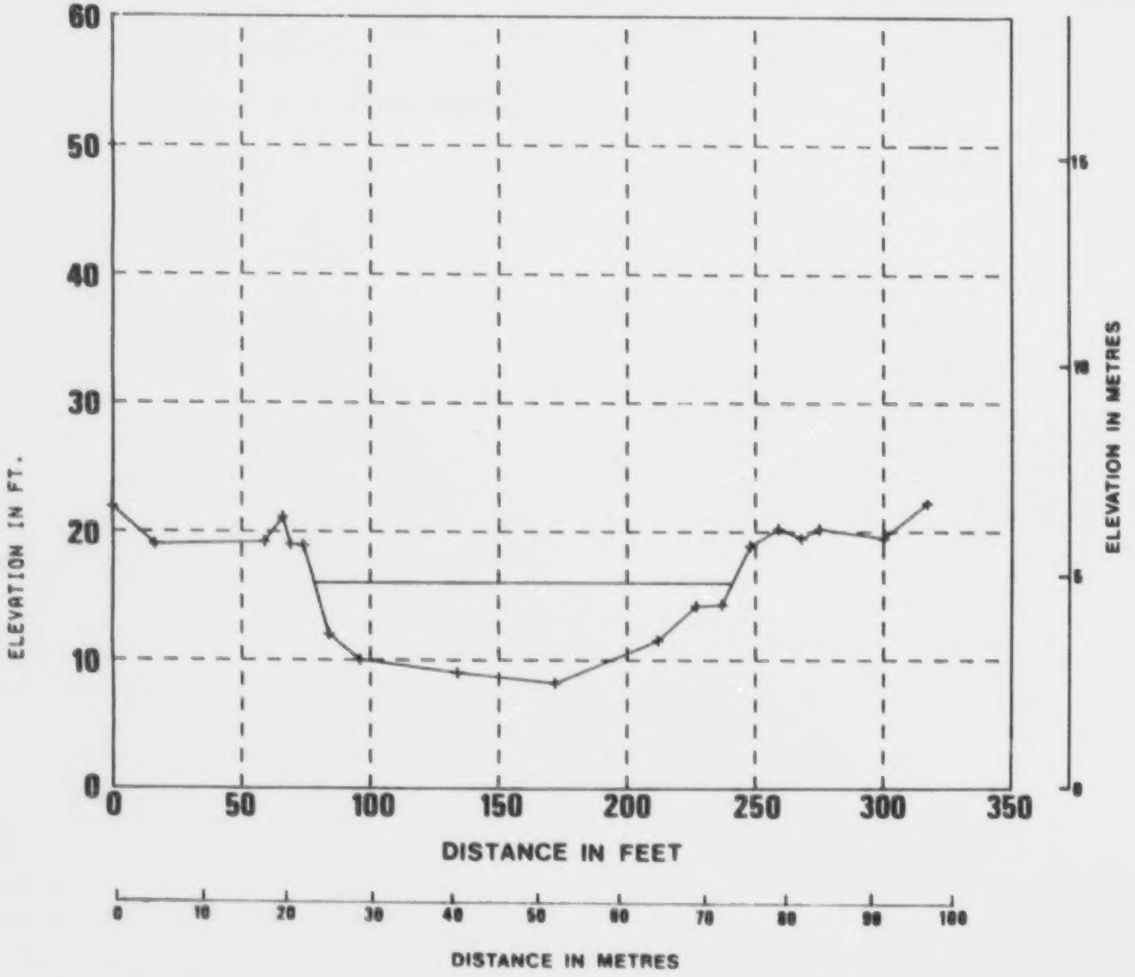
FIGURE 8.2



NORTH RIVER PROFILES
APRIL 29, 1982

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

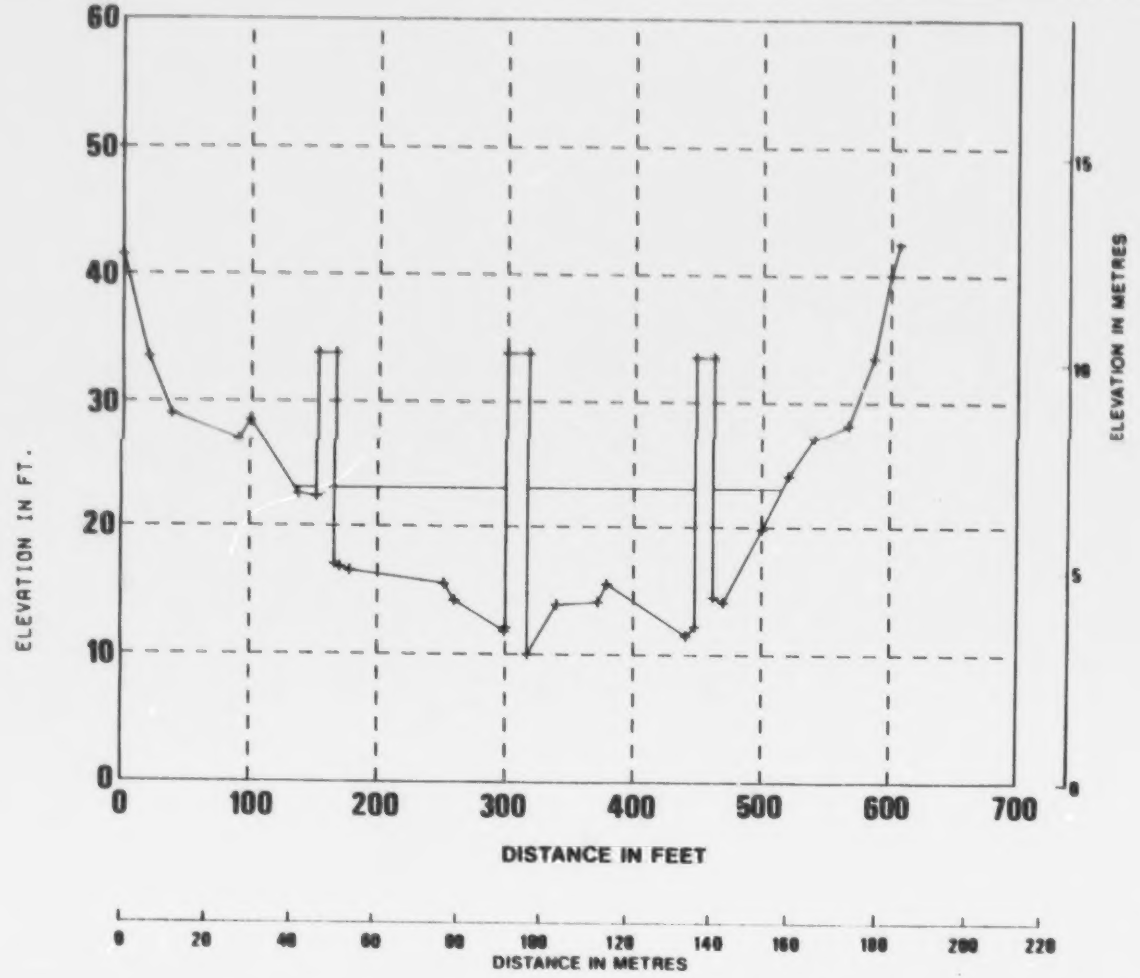
FIGURE 8.3



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

SALMON RIVER AT CONFLUENCE - CROSS-SECTION

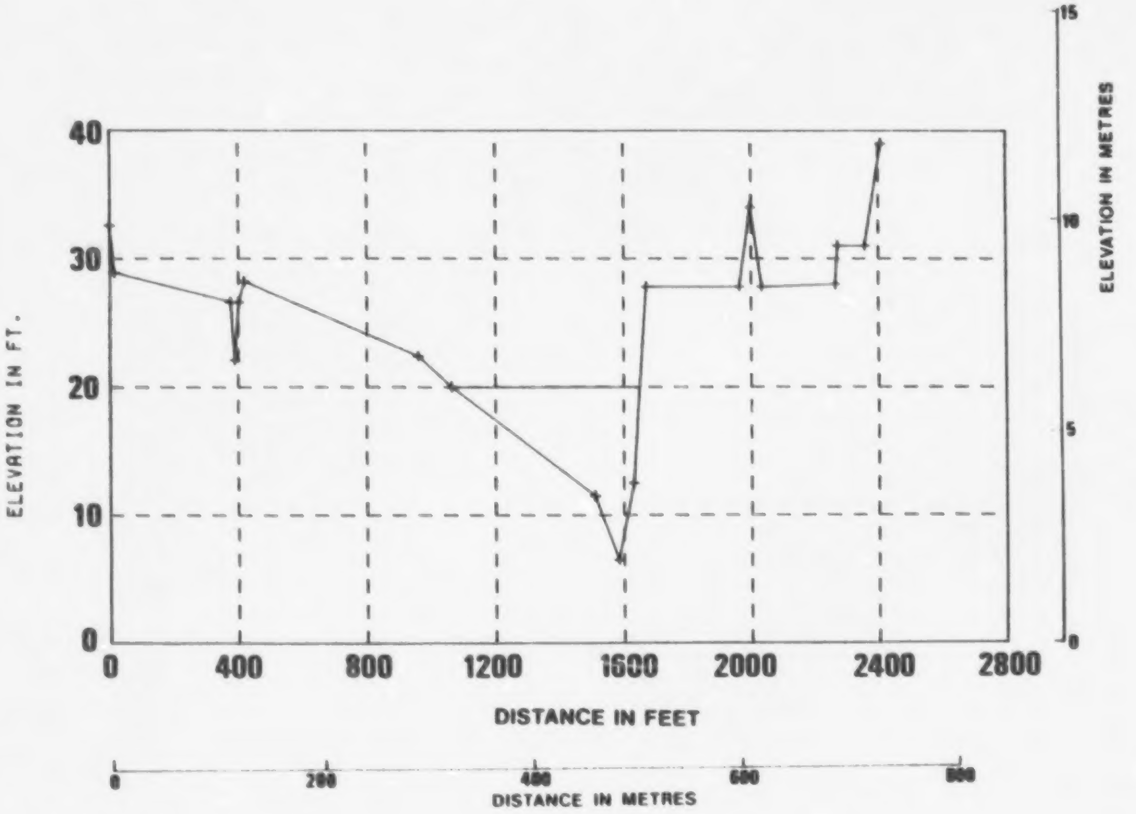
FIGURE 8.4



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

HIGHWAY 102 BRIDGE CROSS SECTION

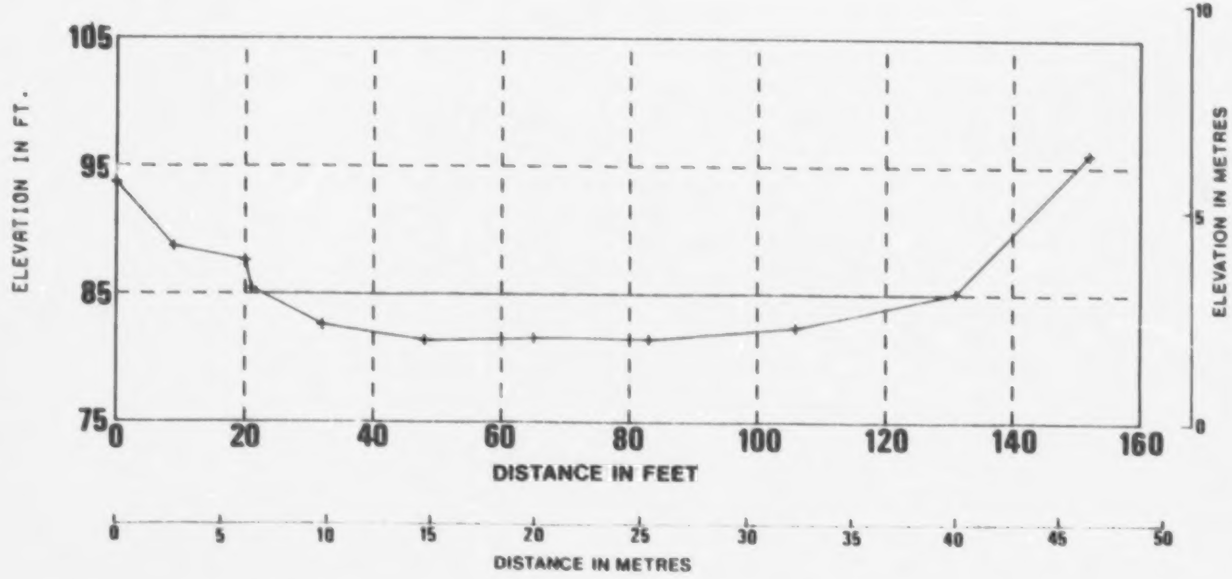
FIGURE 8.5



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

SALMON RIVER
AT OUTLET
CROSS SECTION

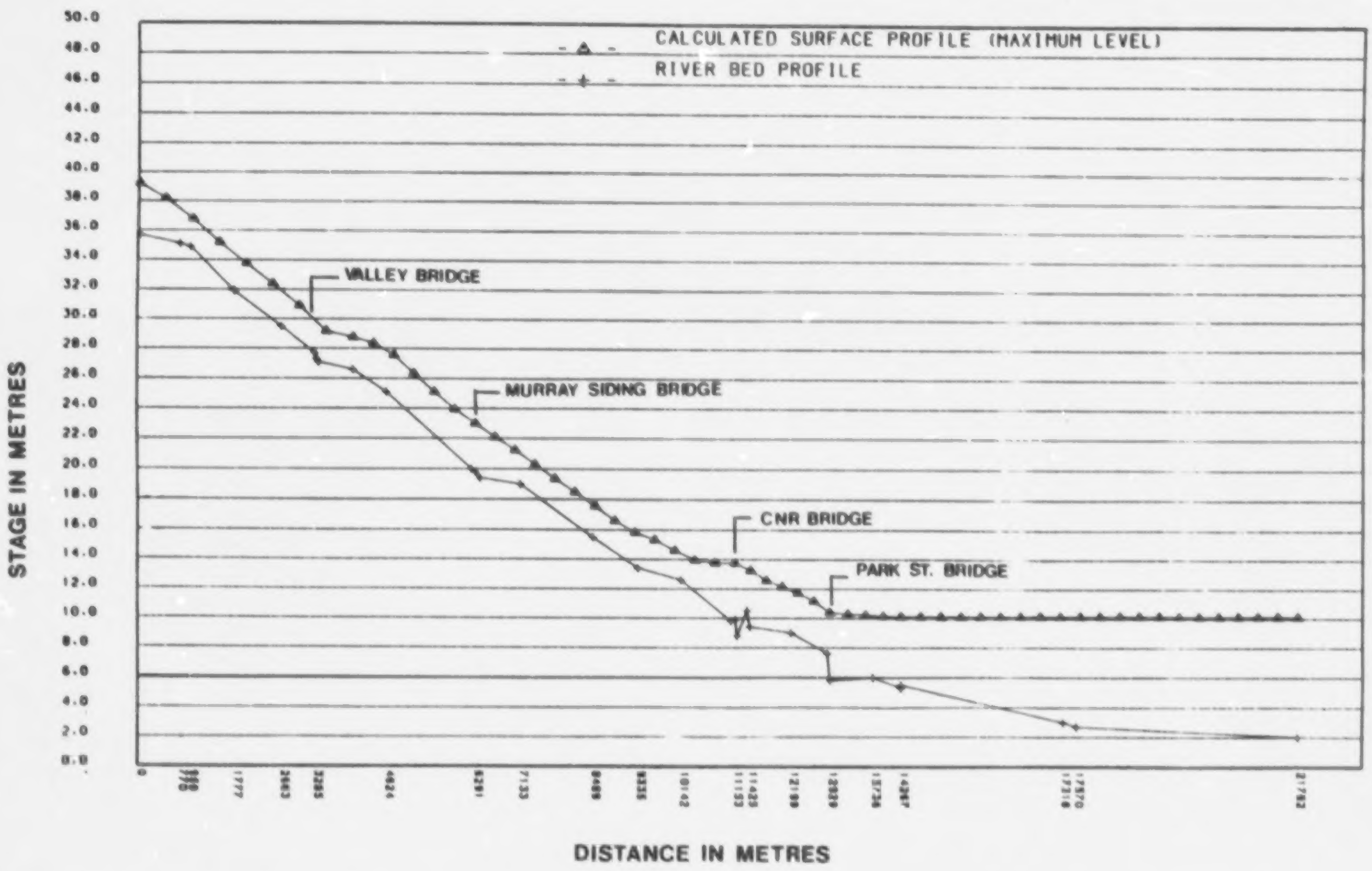
FIGURE 8.6



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

STEWART BRIDGE CROSS SECTION

FIGURE 8.7

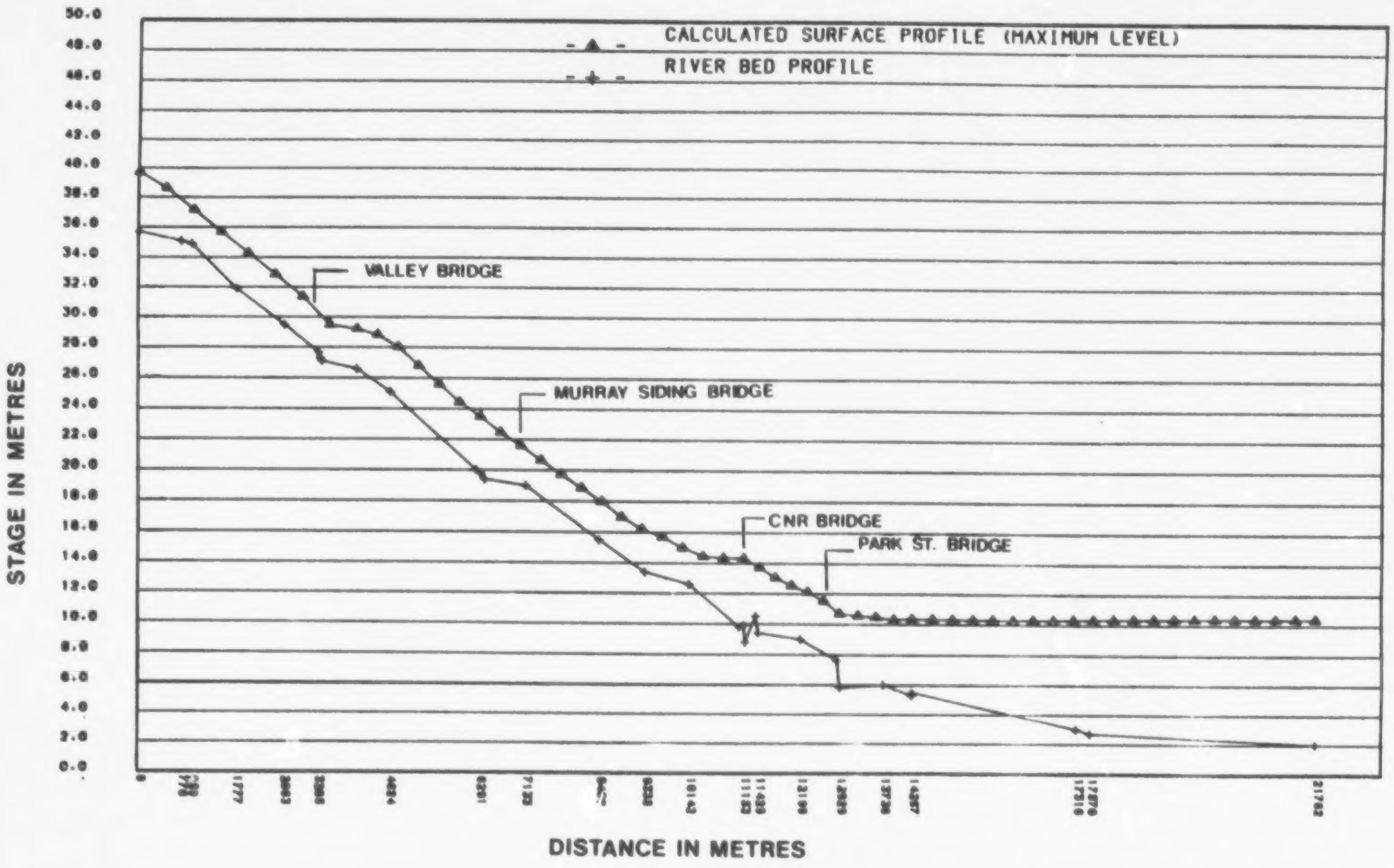


SALMON RIVER FLOOD PROFILE

1:20 YEAR FLOW : 1:20 YEAR TIDE

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 8.8

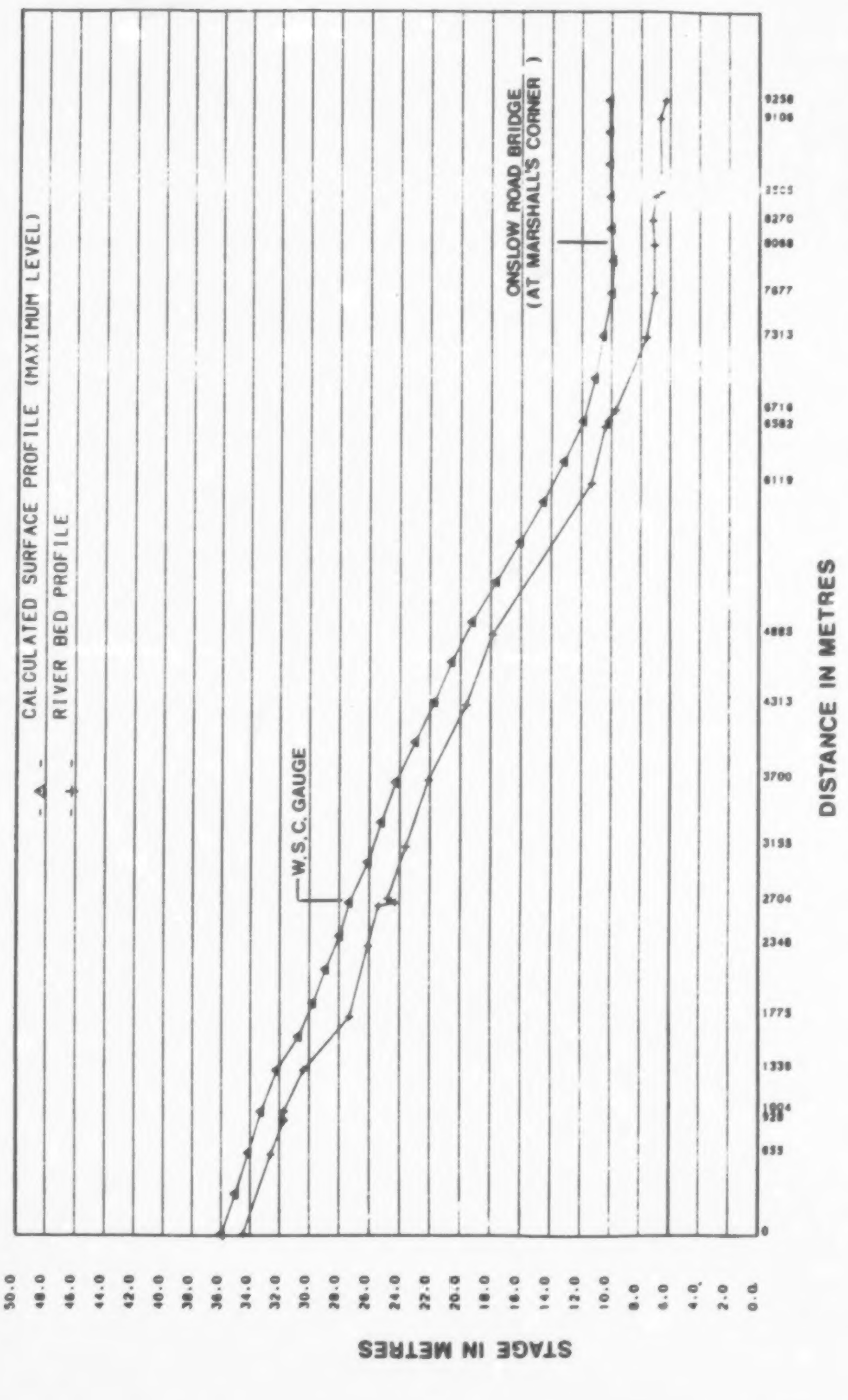


SALMON RIVER FLOOD PROFILE

1:100 YEAR FLOW ; 1:100 YEAR TIDE

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 8.9

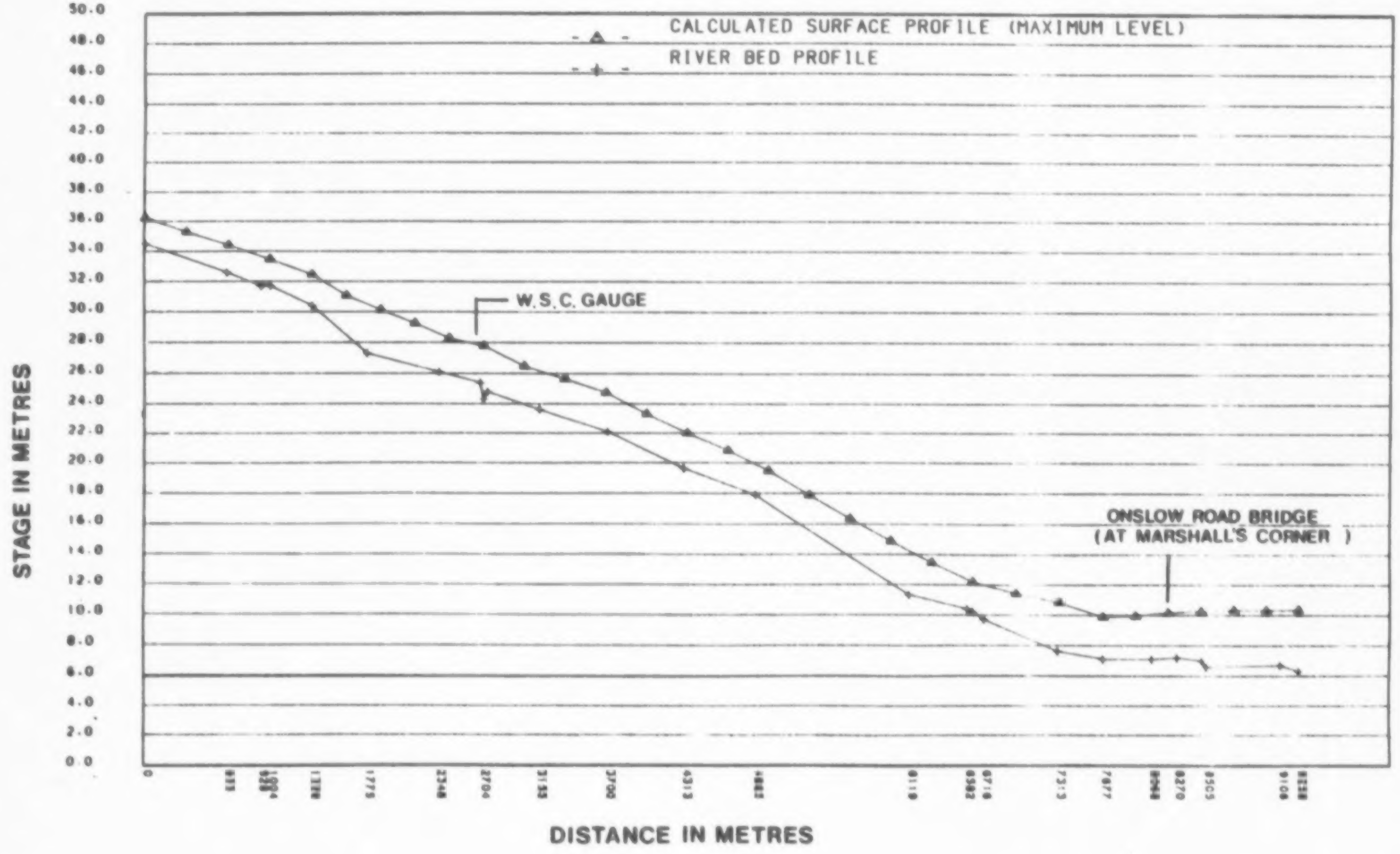


NORTH RIVER FLOOD PROFILE

1:20 YEAR FLOW ; 1:20 YEAR TIDE

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 8.10



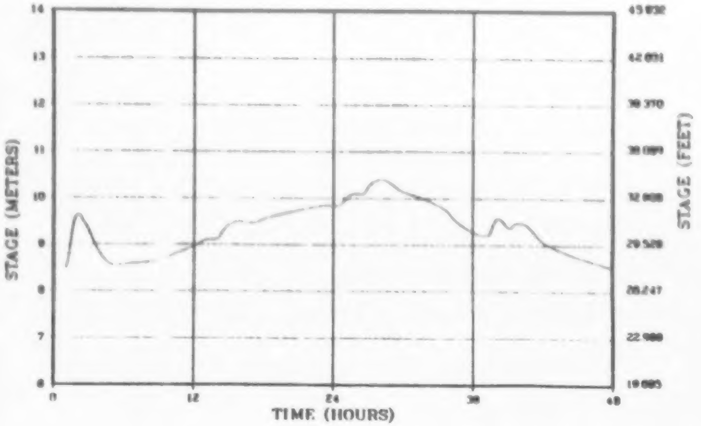
CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

NORTH RIVER FLOOD PROFILE

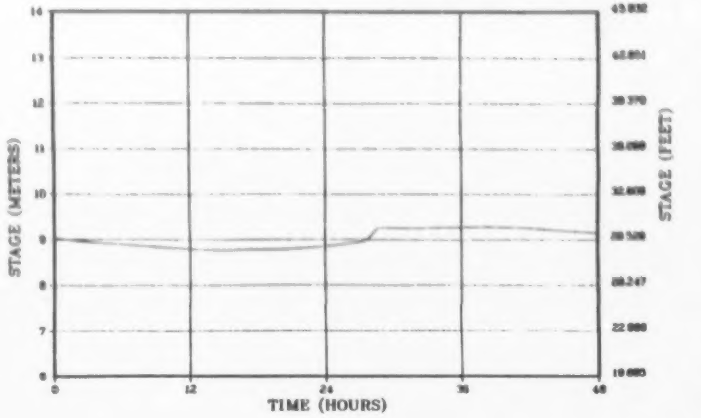
1:100 YEAR FLOW; 1:100 YEAR TIDE

FIGURE 8.11

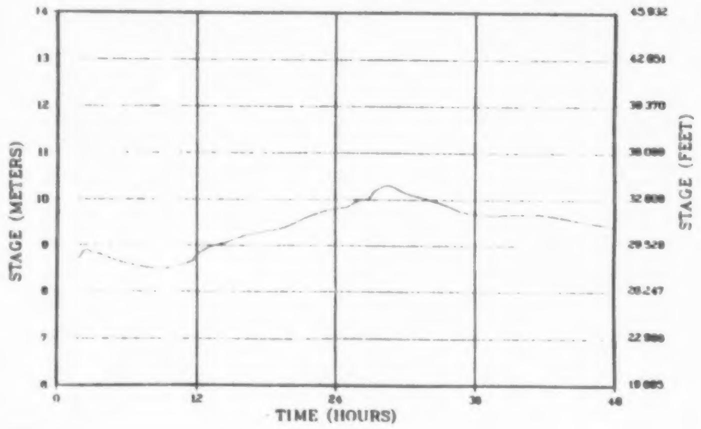
SALMON RIVER AT PARK STREET BRIDGE



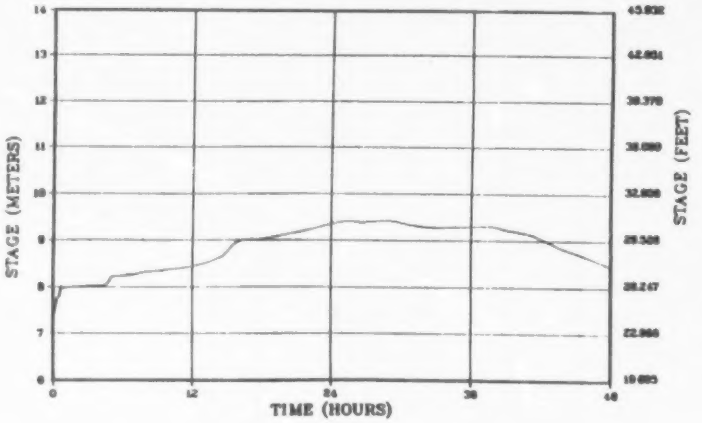
SOUTH FLOOD CHANNEL, PARK STREET BRIDGE.



NORTH SIDE OF FARNHAM BROOK ABOITEAU.



NORTH OF MCCURDY BROOK ABOITEAU.



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

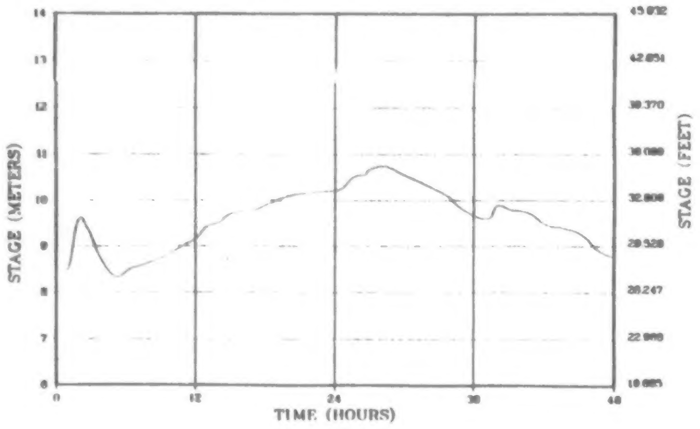
TRURO HYDROTECHNICAL STUDY

SELECTED STAGE HYDROGRAPHS

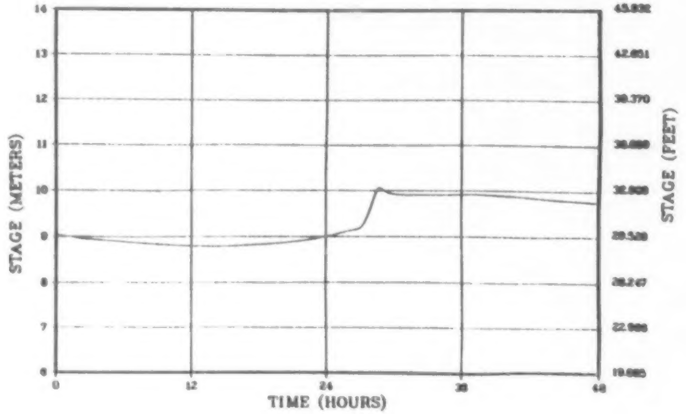
1:20 YEAR FLOW; 1:20 YEAR TIDE

FIGURE 8.12

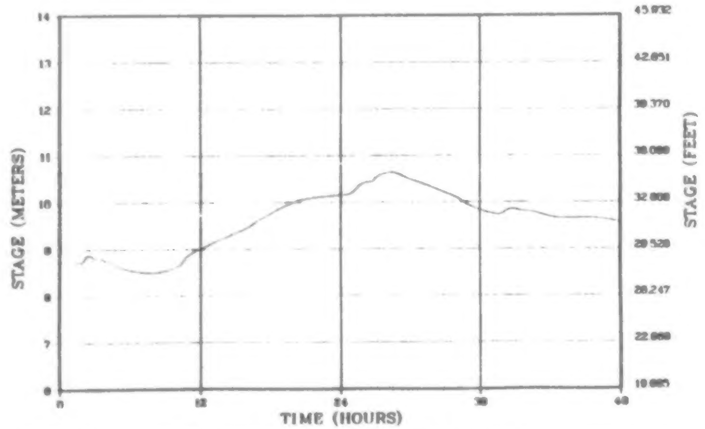
SALMON RIVER AT PARK STREET BRIDGE



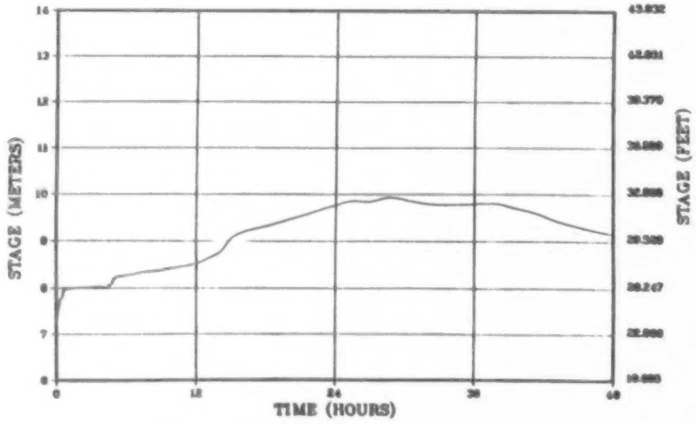
SOUTH FLOOD CHANNEL, PARK STREET BRIDGE.



NORTH SIDE OF FARNHAM BROOK ABOITEAU.



NORTH OF MCCURDY BROOK ABOITEAU.



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

SELECTED
STAGE HYDROGRAPHS
1:100 YEAR FLOW; 1:100 YEAR TIDE

FIGURE 8.13

9.0 REMEDIAL MEASURES

9.1 General

The long history of flooding in the Truro area was discussed in Chapter 3.0. The hydrologic and hydraulic analyses have confirmed the potential for future flood losses and the flood hazard areas have been identified on topographic mapping at scales of 1:1200 and 1:2400. Previous investigations have included proposed measures for mitigating flood damages, and these were summarized in Chapter 4.0.

On the basis of the identification of historical flooding and utilizing the results of the hydrodynamic modelling for the 1:20 and 1:100 year flood profiles, it was possible to identify potential remedial measures for alleviating the flood hazard in the Truro area.

In general, the basic elements of an overall flood damage reduction and remedial works program can be categorized in two main areas:

1. structural measures, which directly affect the flood characteristics, and
2. non-structural measures, which are intended to modify the loss burden, either by reducing the potential for continued development in the flood prone lands (e.g. regulations); or by providing some form of economic relief from flood losses (e.g. relocation or land acquisition).

Detailed investigations and feasibility analyses were beyond the scope of the present investigations. However, it has been possible to identify a number of alternative remedial measures for detailed assessment in the future should such an assessment be conducted.

9.2 Identification of Structural Alternatives

A number of structural measures have previously been identified and considered for the study area (see Chapter 4.0 for example). Both area-wide protective measures and local improvements have been considered. The general location of the various measures discussed are shown on Figure 9.1.

9.2.1 Local Improvements

The historical flood damage centres are listed in Chapter 3.0 and summarized on Figure 3.1. A detailed investigation would be required to estimate mean annual flood damages at each location and to determine if appropriate localized flood protective measures could be adopted in order to provide economical flood protection.

Initially, this could include assessing the effectiveness of recent new ice control berms in the Cottage Street/Lepper Brook area (area 8, Fig. 3.1) and berms in the Riverside/Avon Street area (area 7, Fig. 3.1). This review would be useful in assessing the economic and social benefits of similar work in other damage centres. Similarly, the economic and social impact of house relocation in the Montgomery Drive area could be assessed with a view to establishing the desirability for undertaking a similar program at other damage centres in the future (e.g. possibly in the Stewart Bridge area).

The feasibility of floodproofing existing structures along Robie and Juniper Streets and the requirements for flood-proofing of future structures in this area should be established. Similarly, the feasibility of local flood-proofing and drainage improvements in the Upper Onslow, Marshall's Corner and Farnham Road areas (areas 4, 5 and 6 on Fig. 3.1) could be investigated.

9.2.2 Area Wide Improvements

a) Existing Dykes

The existing dyke system was originally located to provide protection against tidal flood waters. Generally, it does not presently appear to be appropriate to undertake a program to raise dykes to provide flood protection against freshwater flows. For example, near the mouth of the North River, ice is confined to the channel during spring break-up due to constrictions caused by the dyke system. The feasibility of lowering dyke elevations in the Onslow/North River marsh area downstream of the CNR bridge to provide access for the ice to the marsh storage area could, therefore, be investigated.

b) New Dyke Systems

Construction of a new system of dykes in conjunction with channel realignment and improvements, etc. could be given consideration as an area-wide measure for future flood damage reduction. The appropriate flow conveyance capacity (for freshwater floods in conjunction with tides) between the dykes could be determined based on a cost-benefit analysis (essentially from the rail bridges on the North and Salmon Rivers through to the outlet). This would determine the necessary flow conveyance area between the dykes, which would be expected to be significantly greater than for the present system in many areas. This could include straightening the dykes between the confluence and Highway 102 and providing better hydraulic efficiency at the confluence (i.e. the dykes could be set back from the river channel). This could also include the evaluation of the need for a complete realignment of the dyke system to provide optimum hydraulic efficiency for discharge of water and ice.

This assessment could include utilizing the raising of Marshland Drive as part of the new dyke system. This was previously considered (MRMS, 1974), although the problems with local drainage were identified and would have to be addressed as an important part of any future investigations for this and any other dyke system alternatives.

A new trunk sanitary sewer is to be constructed along a right-of-way approximately paralleling Marshland Drive, extending to the Highway 102 viaduct. Construction of a new dyke on this right-of-way could be investigated as an alternative to raising the road.

The possible provision of a new dyke could also be assessed to provide flood protection for existing residences along Salmon River Road (damage location #9 on Figure 3.1). This could be evaluated in conjunction with improvements to the dyke located just downstream (see c(ii) below).

c) Improvements to Existing System

i) The need for improved drainage from the Truro Dykeland Park could be evaluated. Existing flow access to the Highway 102 viaduct is not considered to be adequate. In addition, the outlet capacity is inadequate due to insufficient culvert capacity downstream of the viaduct under Palliser Road and the absence of a suitable flow channel from the viaduct to the Salmon River via the McClures Brook Aboiteau. The benefits of improved flood conveyance capacity, improved drainage via the viaduct, and access to additional storage capacity in the Lower Truro marsh could be evaluated. This could include the provision of additional flow conveyance at Palliser Road (e.g. by lowering the road, etc.) and provision of a flow channel/regrading to the McClures Brook aboiteau.

The drainage improvements could also include evaluation of the need to regrade the area around the McClures Brook viaduct. Over the years, the relocation of the outlet of McClures Brook, the placement of fill along the Brook and the relocation of roads have disturbed the natural drainage features. The requirements for re-establishing efficient drainage could, therefore, be examined in detail.

Finally, in the past it has sometimes been the practice to remove a portion of the dyke near the McClures Brook Aboiteau in order to fully drain the Truro Dykeland Marsh subsequent to the occurrence of flooding. The need for a permanent gated structure, or alternatively, improvements to the McClures Brook Aboiteau, including better access during and after flooding, could be fully investigated.

ii) A new dyke has recently been built in the Cottage/Lepper Brook area (area 8 on Fig. 3.1) for the purpose of reducing ice damages. However, the downstream end of the dyke has not been tied into the railway embankment due to the necessity to provide adequate drainage for the area behind the dyke. As a consequence, floodwaters could still cause damage to structures behind the dyke. The benefits and desirability of providing additional flood protection by completing the dyke and providing an additional drainage system could be evaluated.

d) Ice Control

The analysis of historical flooding and model sensitivity testing have indicated that ice accumulations are a significant factor in flooding in the Truro area.

The upstream movement of estuarine ice could only be effectively controlled by a causeway (see item f).

The structural measures previously discussed could also be evaluated with respect to their ability to reduce ice-related problems. In addition, the feasibility of providing upstream storage areas for riverine ice could be evaluated. For example, the use of areas previously mined for gravel along the North River could be considered for off-channel ice storage. The provision of other measures along the channel to encourage ice movement into off-channel storage areas could likewise be evaluated. For example, the removal of the existing sewage treatment plant near Park Street could enhance the capability of the North River/Farnham Brook Marsh to store ice. The importance of maintaining access to this storage area across Park Street could likewise be confirmed.

e) Flood Control Reservoirs

Flood storage reservoirs located in headwater areas have been proposed in the past in order to control the peak flows to the study area. It is presently considered unlikely that such measures would prove cost-effective and, therefore, future feasibility analyses should likely receive low priority.

f) Causeway/Tidal Dam

Previous investigations have suggested the construction of a causeway/dam to control tides. Eliminating high tides in association with peak freshwater flows would significantly affect flooding in the study area, as confirmed by the model sensitivity analyses. However, the structure would have to be located sufficiently far downstream to provide adequate flood storage in the headpond over a succession of tidal cycles. It is unlikely that provision of such a structure would prove cost-effective and, therefore, any future feasibility investigations should proceed only after the above analyses have exhausted all relevant possibilities.

9.3 Identification of Non-structural Alternatives

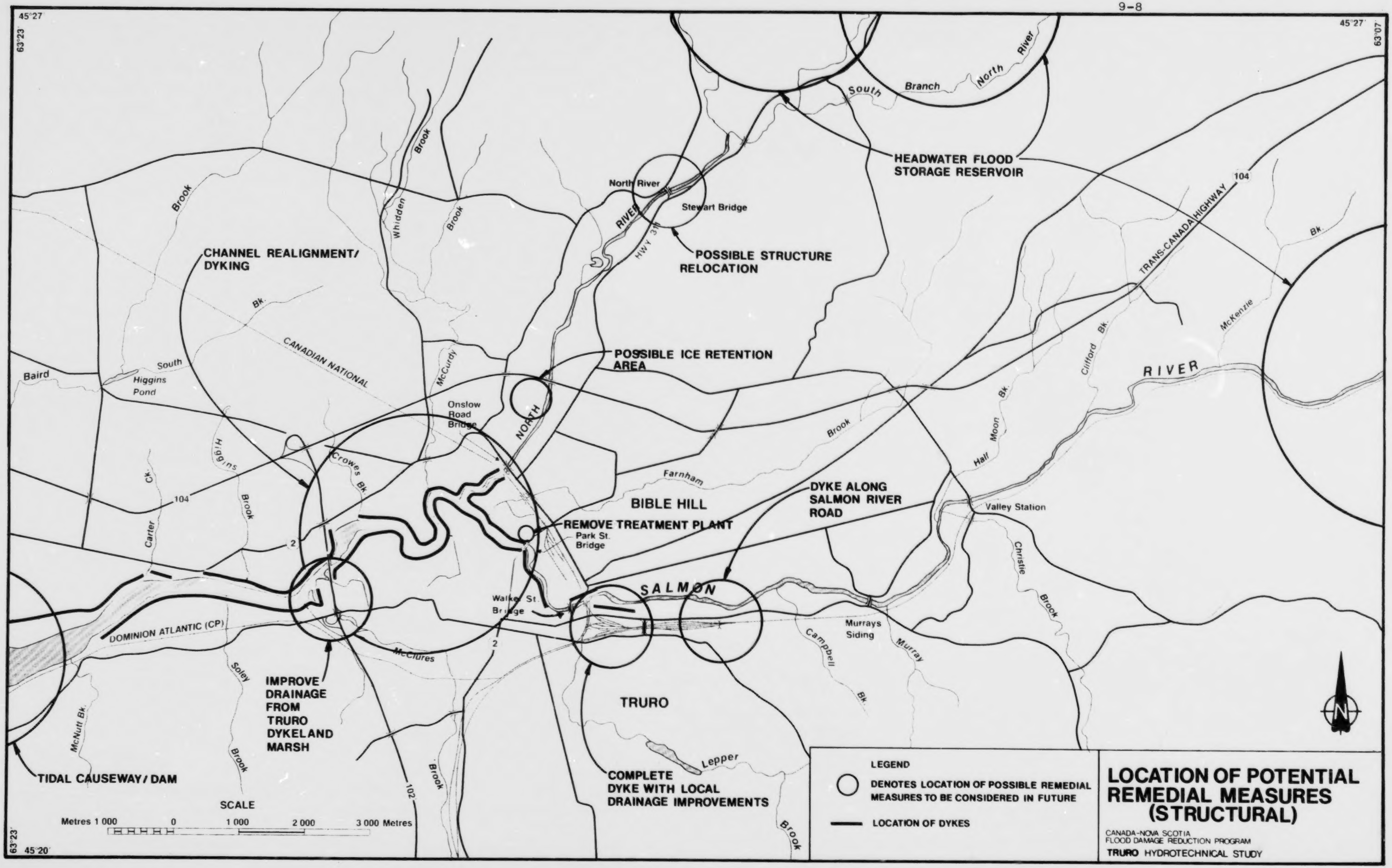
9.3.1 Regulations

Since the area is actively developing, floodplain regulations could be implemented to control future development and thus reduce the potential for continued increases in flood damages. In this case, a two-zone floodway flood-fringe concept is envisaged where zoning regulations would prohibit development in the high hazard floodway. The floodway would be defined as the extent of the 1:20 year flood plain.

The flood fringe area would be defined as the extent of the area between the 1:20 and 1:100 year floodlines. Additional development would be permitted in the flood-fringe areas, depending on the degree of hazard and the implementation of flood proofing measures to protect these developments. Consideration should be given to providing an additional margin of safety.

9.3.2 Other Measures

Other non-structural measures which might be considered include relocation of flood prone structures, payment of flood compensation, purchase of land and development of contingency plans for emergency action.


45°27
63°23
45°27
63°07
HEADWATER FLOOD STORAGE RESERVOIR
South Branch North River
North River
Stewart Bridge
RIVER
HWY 311
POSSIBLE STRUCTURE RELOCATION
CHANNEL REALIGNMENT/ DYKING
Brook
Whidden Brook
Brook
Bk.
CANADIAN NATIONAL
McCurdy
POSSIBLE ICE RETENTION AREA
TRANS-CANADA HIGHWAY 104
Bk.
McKenzie
Clifford Bk.
RIVER
Baird
South Higgins Pond
Onslow Road Bridge
NORTH
Hall Moon Bk.
Brook
Farnham
Crowes Bk.
Higgins
Ck.
Carter
104
Brook
BIBLE HILL
DYKE ALONG SALMON RIVER ROAD
Valley Station
REMOVE TREATMENT PLANT
Park St. Bridge
2
Christie Brook
SALMON
Walker St. Bridge
DOMINION ATLANTIC (CP)
Murrays Siding
Campbell Bk.
Murray
McClures
2
Soley
IMPROVE DRAINAGE FROM TRURO DYKELAND MARSH
TRURO
McNutt Bk.
Brook
Brook
102
Lepper
Brook
TIDAL CAUSEWAY/ DAM
COMPLETE DYKE WITH LOCAL DRAINAGE IMPROVEMENTS
SCALE
Metres 1 000 0 1 000 2 000 3 000 Metres
63°23
45°20
N
LEGEND
○ DENOTES LOCATION OF POSSIBLE REMEDIAL MEASURES TO BE CONSIDERED IN FUTURE
— LOCATION OF DYKES

LOCATION OF POTENTIAL REMEDIAL MEASURES (STRUCTURAL)

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE 9.1

10.0 LIST OF REFERENCES (1)

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DFO = Department of Fisheries and Oceans
DREE = Department of Regional and Economic Expansion
EMO = Emergency Measures Organization
EMR = Energy Mines and Resources
ENV. CAN. = Environment Canada
MRMS = Maritime Resource Management Service
SCS = Soil Conservation Service
U.S. Dept. Agr. = U.S. Department of Agriculture
U.S. Dept. Int. = U.S. Department of Interior

APPENDIX A : BACKGROUND TO FIELD SURVEY PROGRAM

APPENDIX B : BACKGROUND TO HYDROLOGY

APPENDIX C : BACKGROUND TO HYDRODYNAMIC MODELLING

APPENDIX A

BACKGROUND TO FIELD SURVEY PROGRAM

APPENDIX A
BACKGROUND TO FIELD SURVEYS PROGRAM

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APPENDIX A

BACKGROUND TO FIELD SURVEY PROGRAM

A.1 GAUGE READINGS

During the early spring of 1980, the firm of Dillon-Horner Limited installed a network of some 25 staff gauges to record high water levels associated with tides and fresh water flows. Damage by ice and vandals reduced the number of useful gauges over the period of the study, and the locations of these gauges are shown on Figure 5.1.

Observations were taken manually during the periods of high water levels. Observers were continuously present at tide gauge locations while the tide was rising and readings were made frequently (as often as every five minutes) when elevations were changing rapidly. At upstream locations, where changes were not as rapid, hourly readings were obtained.

Observations of water elevations were made by the consultant's staff except for the event of 29 April, 1982, when a team of provincial and federal civil servants, under the direction of the Technical Committee, made the observations. The occasions when observations were made at each gauge are indicated on the matrix diagram, Table A-1.

The water level data collected for the events of 1980, 1981 and 1982 are summarized on Figures A-1 to A-16 (geodetic elevations).

TABLE A.1

Summary of Occasions When Gauges were Read

		March/80					April/81		Nov. 1981		Dec.	Feb. 1982	Apr.
		15	16	17	18	19	16	17	11	23	16	12	23
Bass River	SB						o	o	o	o	o		
Cutten Brook	S1	o	o	o	o	o	o	o	o	o			
McClures Aboiteau	S2	o	o	o	o	o				o		o	o
Board Landing Bridge	S3	o	o	o		o	o	o	o	o	o		o
McCurdy Brook	S4						o						o
Farnham Brook	S6												o
Park St. Bridge	S7						o	o	o	o	o	o	o
CNR Bridge	S8												o
Gillis	S9												o
Murray Siding Bridge	S10												o
Valley Bridge	S11												o
Clifford Bridge	S12												o
CNR Aboiteau	N1												o
Bailey Bridge	N2												o
McClures Brook	MC												o

A.2 CROSS-SECTIONS

A.2.1 Methodology

In order to determine the shape of the cross sections and the slope of the river channels, extensive surveys were done in the summer of 1982. A tachymeter theodolite was used by a technologist with the Nova Scotia Department of the Environment. With this instrument, use was made of the stadia principle and trigonometry to measure distances, and differences in vertical elevations to geodetic elevations.

Cross-sections were selected at representative locations and channel constrictions to define the character of the river channels. All sections were identified on aerial photographs and photographs were taken into the field by the surveyors for use in identifying the cross-section locations. The cross-section locations are shown approximately on Figure 5.1.

Generally the procedure used in measuring a cross-section was to set the instrument at one end of the cross-section and read stadia distances and vertical angles to a reference point and to each point of interest on the section, e.g. thalweg, water's edge, toe of bank, top of bank, etc. In some situations, either because sight lines prevented it or because two sections were visible from one set up, two sections were "shot" from a single location. In such cases, of course, the horizontal angle between successive points was recorded and used to obtain the required distances. Whichever procedure was used, a reference point for elevations to which all points in the cross-section were related, was established and marked for future tie-in of the cross-section and water level gauges to geodetic datum by spirit levelling procedures.

Level surveys were undertaken by one of two methods:

1. from one geodetic bench mark to another (published by GSC) with the reference points for cross-sections and gauges being included in the run (the "differential levelling" method)
2. circuits run from a geodetic bench mark to the reference point and back to the point of starting (the "closed circuit" method)

All the geodetic bench marks were considered reliable and had been checked by Nova Scotia Department of Environment staff and by other surveyors, for various jobs, including river works and basic mapping for the Nova Scotia Department of Municipal Affairs and for this project. Table A.2 lists the geodetic bench marks used for the levelling.

The levelling run between Murray and Union indicated that the Union Bench mark was suspect with a closure error of 0.1132 m. However, a return run closed with only a small error of 0.0299 m when brought back to its starting point at the Murray Siding Bridge. (Union is a second order benchmark set in 1907). The Union benchmark was, therefore, not used in the study. Closure errors for circuits from and to a local geodetic benchmark were frequently in the order of .0006 m with the greatest being .0088 m (see Figure 5.1).

Cross-section data for the estuary was measured earlier by Environment Canada and the predecessor of the Maritime Resource Management Service (M.R.M.S.), of Amherst N.S. The most seaward section (SA 34) was surveyed by M.R.M.S. while those between it and the confluence of the Salmon River were by Inland Waters/Lands Directorate, Environment Canada.

A.2.2 Check Surveys

In checking the survey results with topographic mapping, an error became evident in relation to the elevation of cross-section SA-17. Further, in the winter of 1985 it was found that the computer

TABLE A.2

List of Geodetic Bench Marks used for Truro Area Levelling

69N205	Princess Margaret Rose School, Willow Street
#2041	Victoria Square, end of Willow St. Fundamental BM
536-B	Bank of Nova Scotia building Prince St. at Inglis
535-B	Office building, corner Lorne and Prince Streets
NS6956	concrete, SW side of Hwy. #311 0.2 km SW of North River bridge
NS6961	concrete, SW side of Hwy. #311 30 m SW of red barn
NS6999	concrete, side of Hwy. #311 2.6 km NW of junction Hwy. #4
69N158	Hwy. #102 bridge 0.2 km South of bridge over Salmon River
69N191	on Ponderosa tavern foundation
69N199	#102 Hwy. bridge over route #311
NS6952	south side of Hwy. #311, 1.3 km NE of highway bridge over North River
NS6962	concrete, side of Hwy. #311, 2.1 km north of jet of Brookside Road
69N148	Riversdale Lumber Co. office, east of Queen Street
69N149	Meadowvale Dairy (Farmers Co-Op), north side of Queen Street, 3.4 km east of R.R. Station
69N198	Wall of building at Marshall's Corner, end of Hwy. #311
69N157	Hwy. #102 bridge over DAR track and old road, 3.5 km west of R.R. Station
UNION CMLXXXVII	CNR stone box culvert, 183 m west of station, bolt in south face (not used)
WSC	Plug top of north abutment Murray Siding Bridge, downstream side

TABLE A.2 (cont'd)

NS7006	NE side of Main Street, south of Farnham Brook
NS23899	Hydrant in front of #5 (sewage) pumping station, Lorne Street, near Stadium

NOTE: See Figure 5.1 for approximate locations of Bench Marks used in undertaking field surveys.

model was not giving reasonable results in the vicinity of the Murray Siding Bridge. Therefore, it was judged necessary to check some survey results, and accordingly, an independent firm of Nova Scotia Land Surveyors was contracted to resurvey sections SA-12, SA-13, SA-14, and SA-17 (Salmon River) and NR-9, NR-10, NR-13 and NR-19 (North River) (Hingley, G., 1985). These locations were identified on the same aerial photographs used by the original surveyors to identify the cross-section locations. The check-surveys were done in August of 1985.

At each section a TBM was set for reference by spirit levelling. This was related by differential levelling to geodetic benchmarks levelled and published by Geodetic Survey of Canada. End points of each section were staked and their elevations established by reference to the TBM using spirit levelling procedures. Intermediate points were observed using a Wild Model T-2 theodolite and double-faced metric levelling rod. Thus, two separate observations for each point were made, providing a check on the readings. Horizontal and vertical distances for each point on the section were determined by stadia principle. The spirit levelling of the end points provided a check on the stadia method. The observations were reduced and plotted. Closing errors in the spirit levelling circuits were generally about .0008 m. For the runs between geodetic benchmarks, errors were not more than .0034 m. This work confirmed assumptions of an error in surveying of section SA-17. The new data was used in the model for this section and for sections near Murray Siding Bridge. The check survey confirmed the original surveys on the North River and the remainder of the Salmon River.

The levelling accuracy needed for hydraulic modelling is relatively low (third order levelling), but the specifications for the check survey set a stringent maximum (in metres) of $.0082 \sqrt{L}$, where L is the survey distance in km, as the allowed closure in elevations. This corresponds to second order accuracy. The bulk of the original

A.2.3 Cross-section Plots

The cross-section locations are shown on Figure 5.1. Identification codes relating the field sections and the model input structure (see Section 8.0) are listed in Table A.3. A complete set of computer plotted cross-sections is available as part of the supplementary background information available for this project.

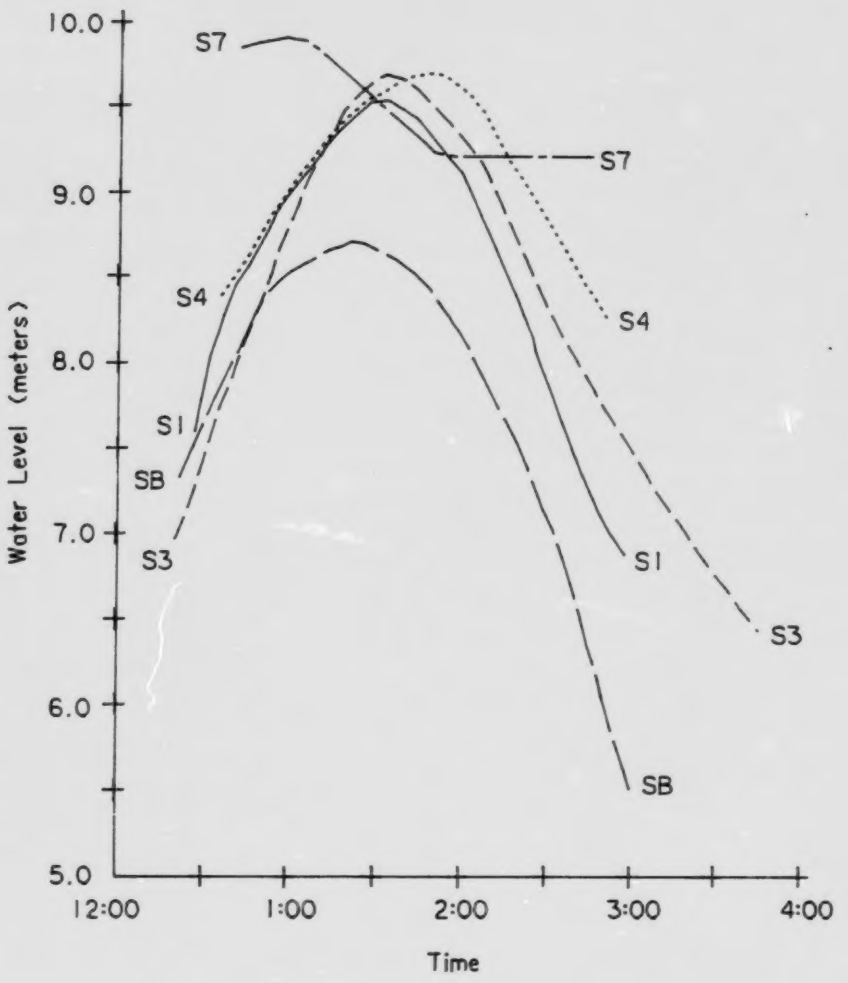
The location of field surveyed sections are identified on Figure 5.1. A cross-reference index between field surveyed section identification numbers and the identification numbers used in the computer model is given in Table A.3. In some instances a single field surveyed section was found to be representative of the cross-sections at several locations and was utilized accordingly.

The field surveys were also supplemented by deriving cross-sections from topographic maps at a scale of 1:1200 and 1:2400 (0.5 m contour interval). This was done primarily in the flood channel areas (e.g. dotted lines on Figure 5.1) and these cross-section plots are also included in the supplementary computer plots of all cross-section data.

TABLE A.3

Reference Table for Cross-sections

Surveyed	Modelled
NR-1B	R10-1
NR-1A	R10-2
NR-1	R10-3
NR-2	R10-4
NR-3	R10-5
NR-4	R10-6, R30-3, R40-1
NR-5A	R30-1
NR-5	R30-2
NR-6	R40-2
NR-7	R40-3
NR-8	R40-4
NR-9	R40-5, R50-1
NR-10	R50-2
NR-11	R50-3
NR-12	R50-4
NR-13	R50-5
NR-14	R50-6
NR-15	R50-7
NR-16	R50-8
NR-17	R50-9, R60-1
NR-18	R60-2
NR-19	R60-3, R70-1
NR-20	R60-4, R70-2
NR-21	R70-3
NR-23	R70-4
NR-24	R70-5
NR-25A	R70-6
NR-26	R70-7
SA-2	R120-1
SA-3	R120-2
SA-4	R120-3
SA-5	R120-4
SA-6	R120-5
SA-7	R120-6
SA-8	R120-7
SA-9	R120-8
SA-10	R120-9, R130-1
SA-11	R130-2
SA-12	R130-3
SA-13	R130-4, R140-1
SA-14	R140-2
SA-15	R140-3
SA-16	R140-4
SA-17	R140-5
SA-18	R140-6
SA-19	R140-7
SA-20	R140-8, R150-1
SA-21	R150-2
SA-22	R150-3
SA-24	R150-4
SA-25	R150-5
SA-26	R150-6
SA-27	R150-7, R160-1
SA-29	R160-2
SA-30	R160-3
MC230-4	R230-1
MC230-5	R230-2
MC230-7	R230-3, R240-1
MC240-8	R240-2
MC240-8A	R240-3
MC240-9	R240-4
MC250-10	R250-1
MC250-11	R250-2
MC250-12	R250-3
MC250-13	R250-4
MC250-14	R250-5
SA31	R270-1
SA32	R270-2
SA33	R270-3
SA34	R270-4



NOTE:
S1= STAFF GAUGE NUMBER;
SEE FIGURE 5.1 FOR LOCATION

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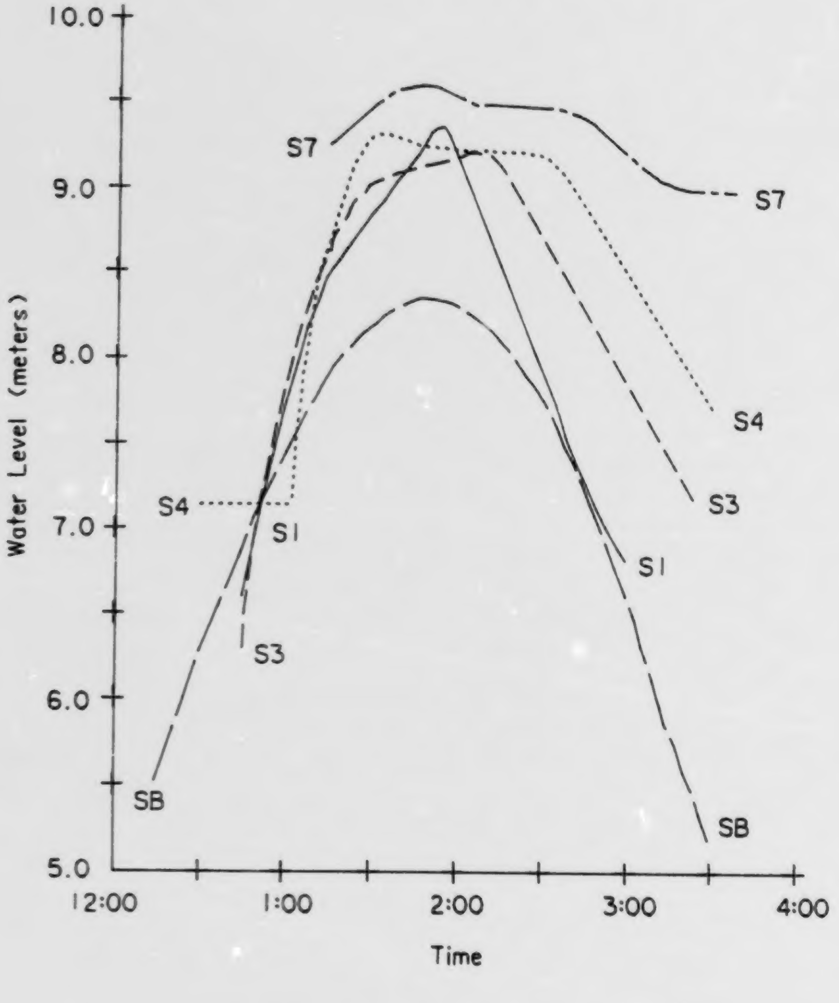
TRURO HYDROTECHNICAL STUDY

TIDAL LEVELS OBSERVED APRIL 16, 1980. - A.M.

FIGURE A.1



NOTE:
S1= STAFF GAUGE NUMBER;
SEE FIGURE 5.1 FOR LOCATION

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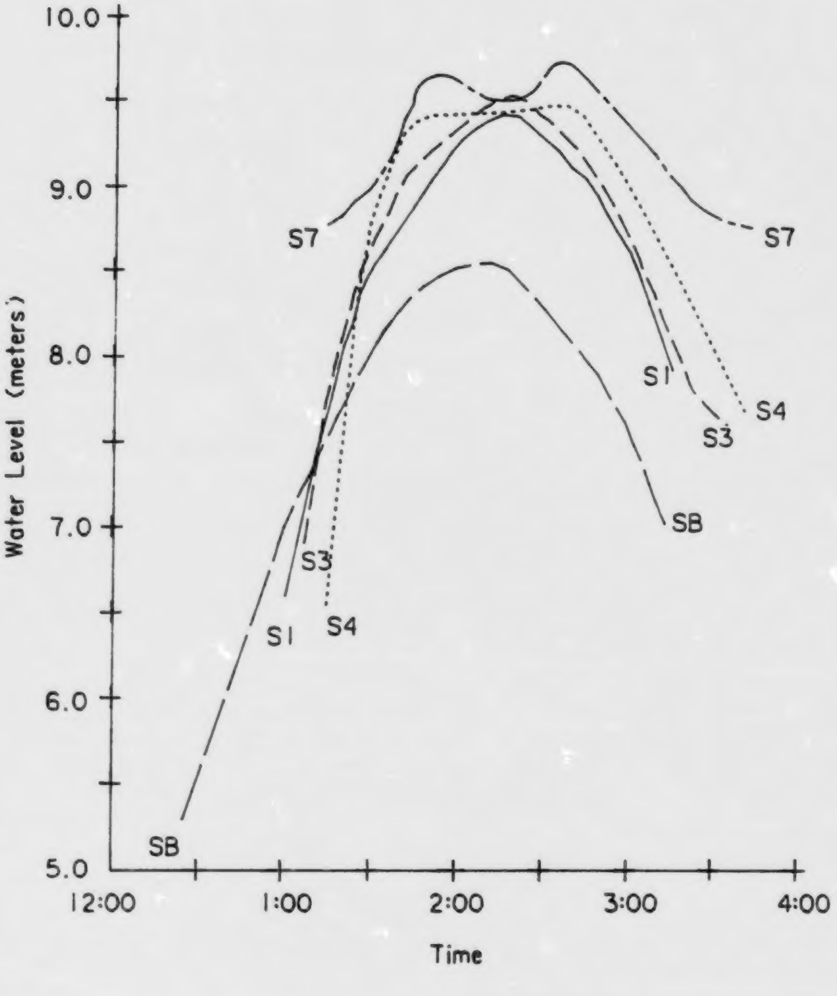
TIDAL LEVELS OBSERVED APRIL 16, 1980. - P.M.

FIGURE A.2

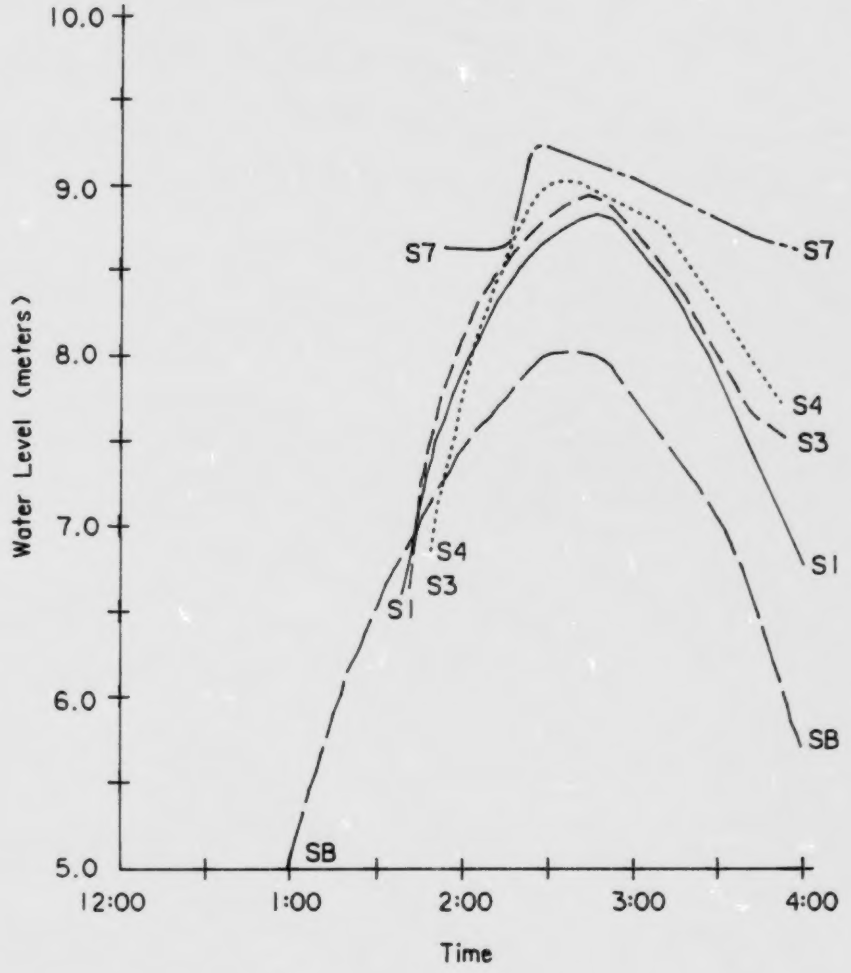


NOTE:
S1= STAFF GAUGE NUMBER;
SEE FIGURE 5.1 FOR LOCATION

TIDAL LEVELS OBSERVED APRIL 17, 1980. - A.M.

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE A.3

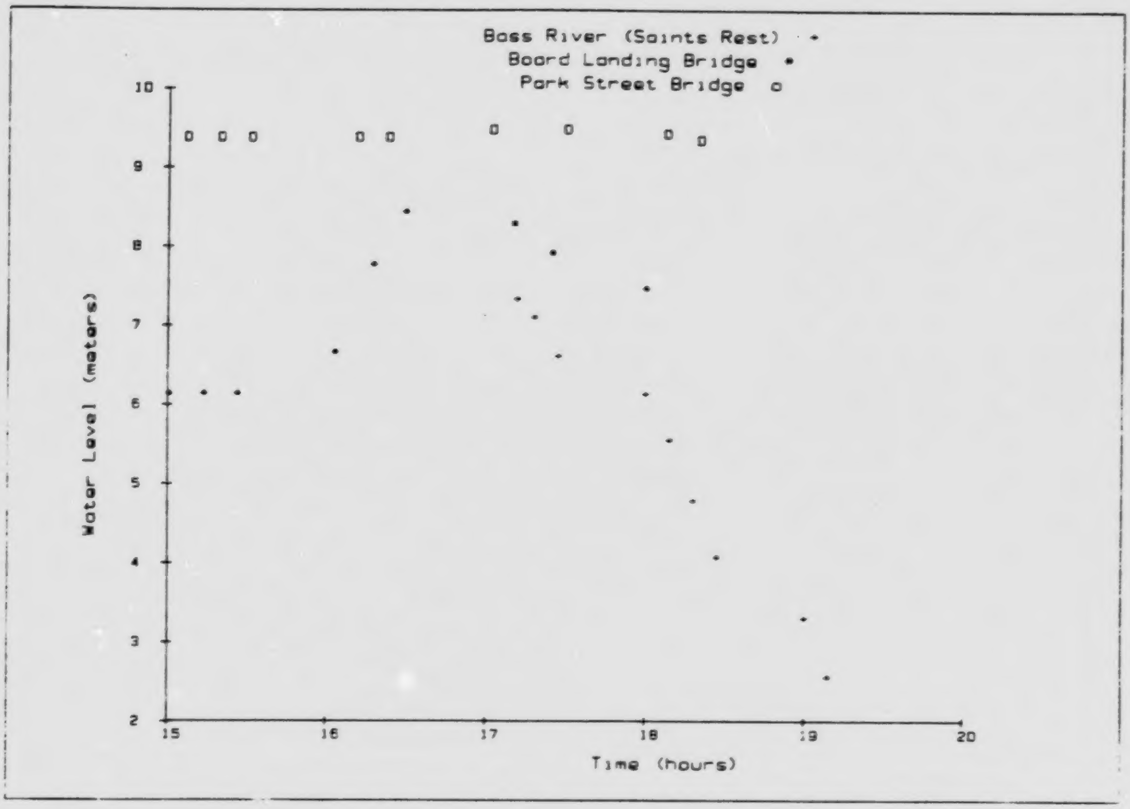


NOTE:
S1= STAFF GAUGE NUMBER;
SEE FIGURE 5.1 FOR LOCATION

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

TIDAL LEVELS OBSERVED APRIL 17, 1980. - P.M.

FIGURE A.4



Bass River (Saints Rest)

Hour	Water Level
17.19	7.33
17.30	7.10
17.45	6.61
18.00	6.13
18.15	5.55
18.30	4.78
18.45	4.07
19.00	3.30
19.15	2.56

Board Landing Bridge

Hour	Water Level
15.01	6.14
15.23	6.14
15.44	6.14
16.05	6.66
16.29	7.77
16.49	8.44
17.17	8.29
17.41	7.92
18.00	7.47

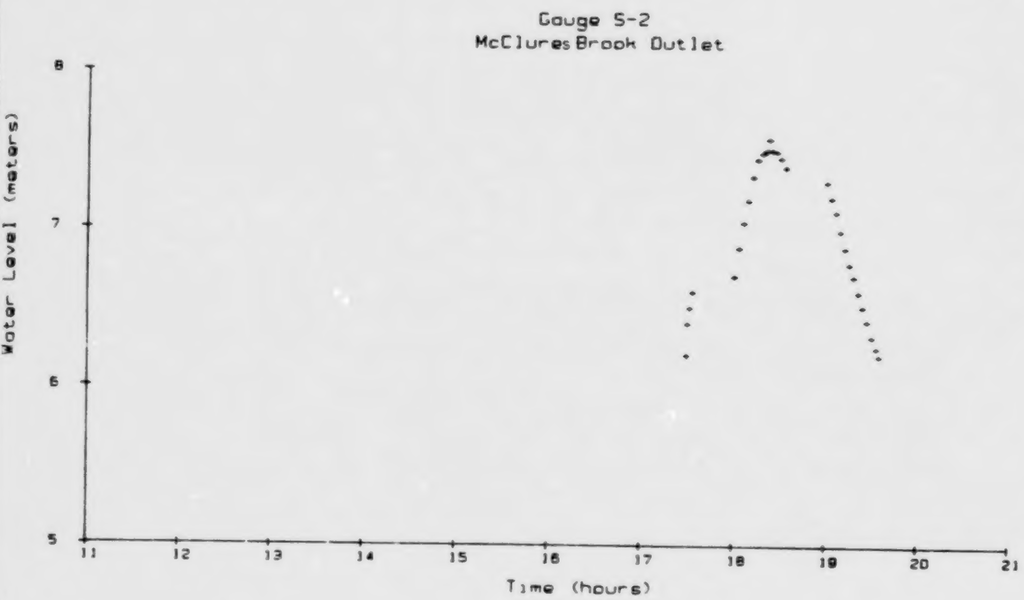
Park Street Bridge

Hour	Water Level
15.12	9.38
15.33	9.38
15.52	9.38
16.19	9.38
16.38	9.38
17.03	9.48
17.50	9.48
18.13	9.43
18.34	9.35

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

TIDAL LEVELS OBSERVED DECEMBER 16,1981 - P. M.

FIGURE A.5



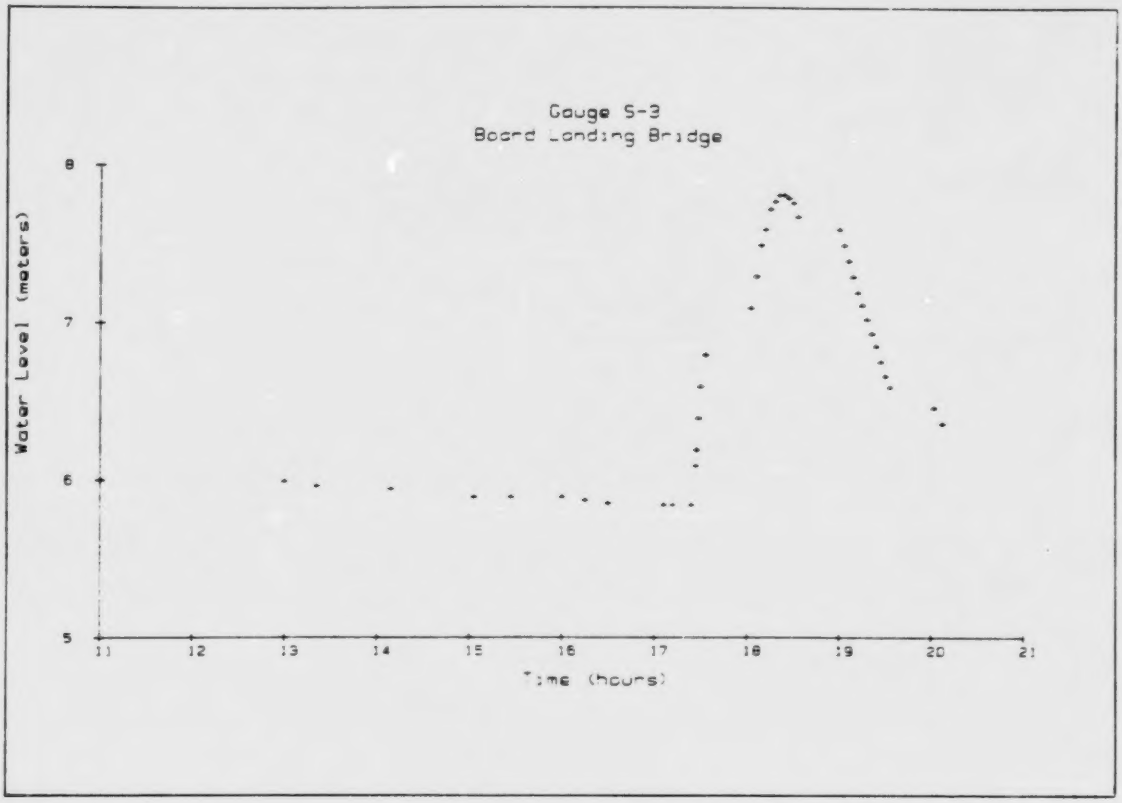
Hour	Water Level	Hour	Water Level
17.49	6.21	18.50	7.46
17.50	6.41	18.55	7.40
17.52	6.51	19.00	7.31
17.55	6.61	19.05	7.21
18.00	6.71	19.10	7.12
18.05	6.89	19.15	7.00
18.10	7.05	19.20	6.89
18.15	7.19	19.25	6.79
18.20	7.34	19.30	6.71
18.25	7.45	19.35	6.61
18.30	7.49	19.40	6.52
18.35	7.51	19.45	6.43
18.37	7.58	19.50	6.33
18.40	7.51	19.55	6.26
18.45	7.50	19.58	6.21

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS McCLURES BROOK

APRIL 29, 1982

FIGURE A.6



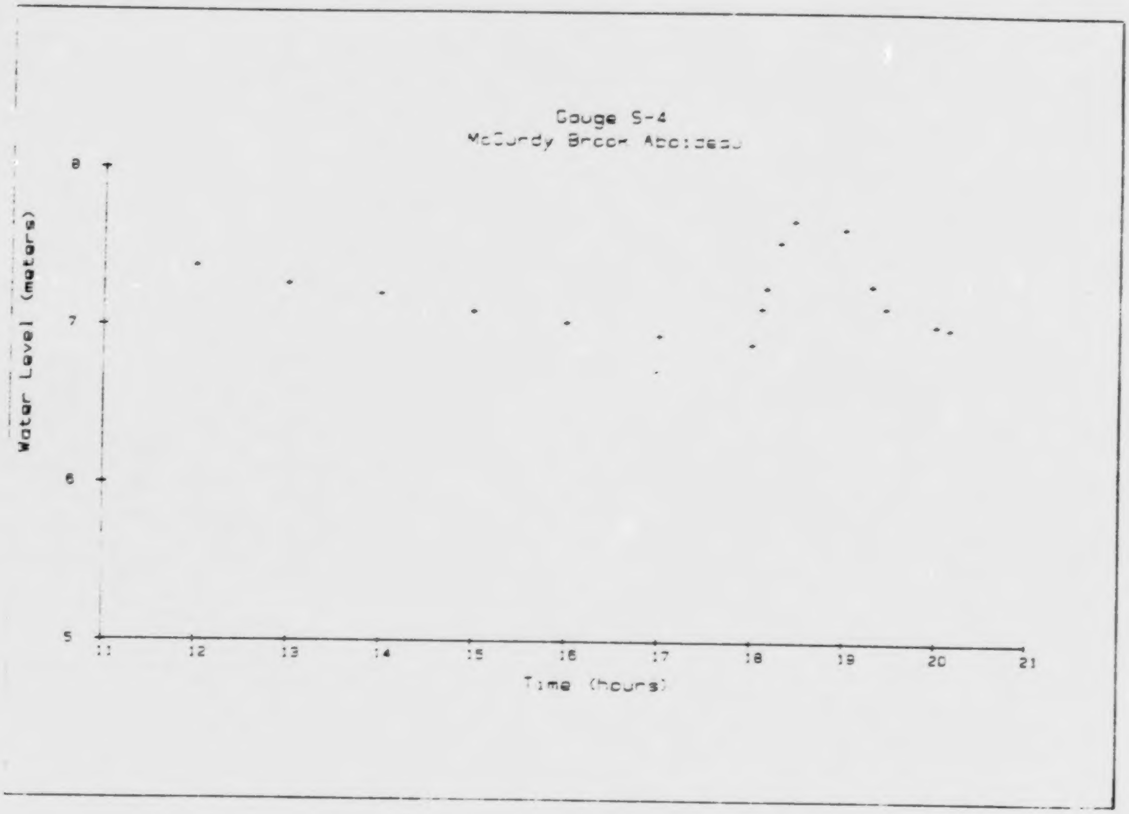
Hour	Water Level	Hour	Water Level
13.35	5.96	18.30	7.77
14.15	5.94	18.35	7.81
15.05	5.89	18.40	7.81
15.45	5.89	18.45	7.79
16.00	5.89	18.50	7.76
16.25	5.87	18.55	7.67
16.50	5.85	19.00	7.59
17.10	5.84	19.05	7.49
17.20	5.84	19.10	7.39
17.40	5.84	19.15	7.29
17.45	6.09	19.20	7.19
17.46	6.19	19.25	7.11
17.48	6.39	19.30	7.02
17.50	6.59	19.35	6.93
17.55	6.79	19.40	6.85
18.04	7.09	19.45	6.75
18.10	7.29	19.50	6.66
18.15	7.49	19.55	6.59
18.20	7.59	20.03	6.46
18.25	7.72	20.12	6.36

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS BOARD LANDING BRIDGE

APRIL 29, 1982

FIGURE A.7



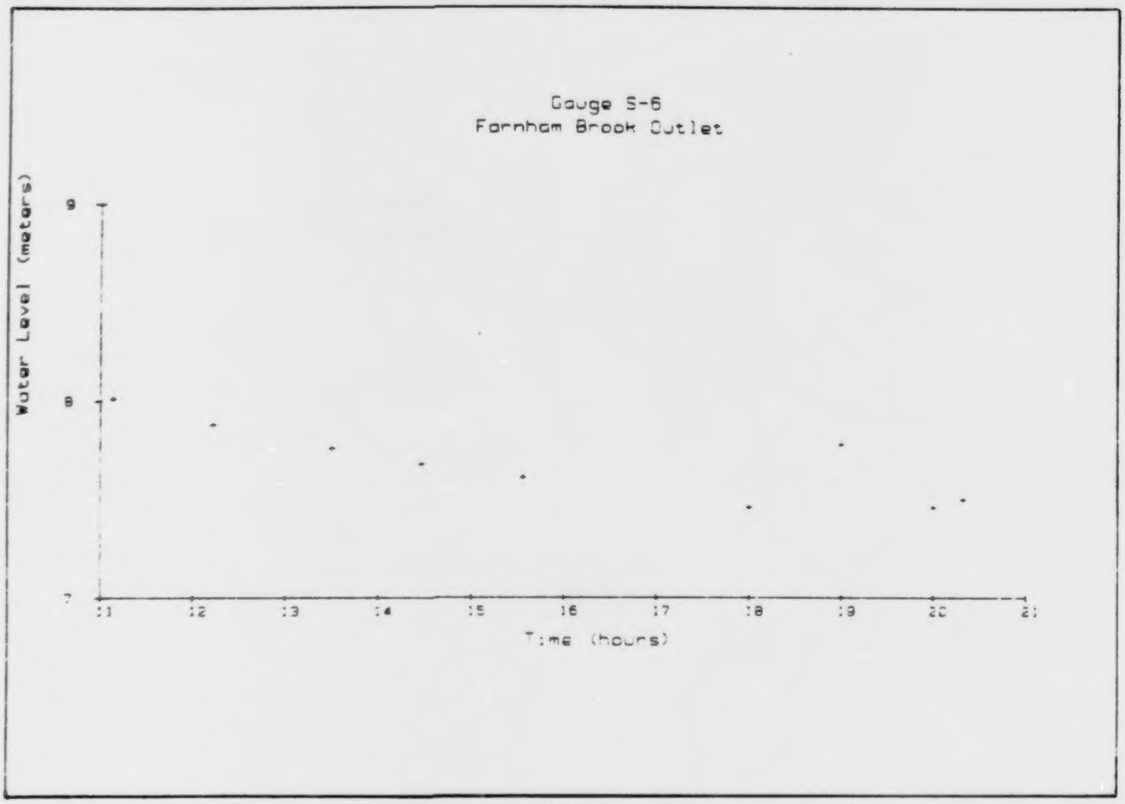
Hour	Water Level
12.00	7.37
13.00	7.26
14.00	7.20
15.00	7.09
16.00	7.02
17.00	6.94
18.00	6.89
18.10	7.12
18.15	7.25
18.30	7.54
18.45	7.68
19.00	7.63
19.30	7.27
19.45	7.13
20.00	7.02
20.15	7.00
20.50	5.50

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS McCURDY BROOK

APRIL 29, 1982

FIGURE A.8



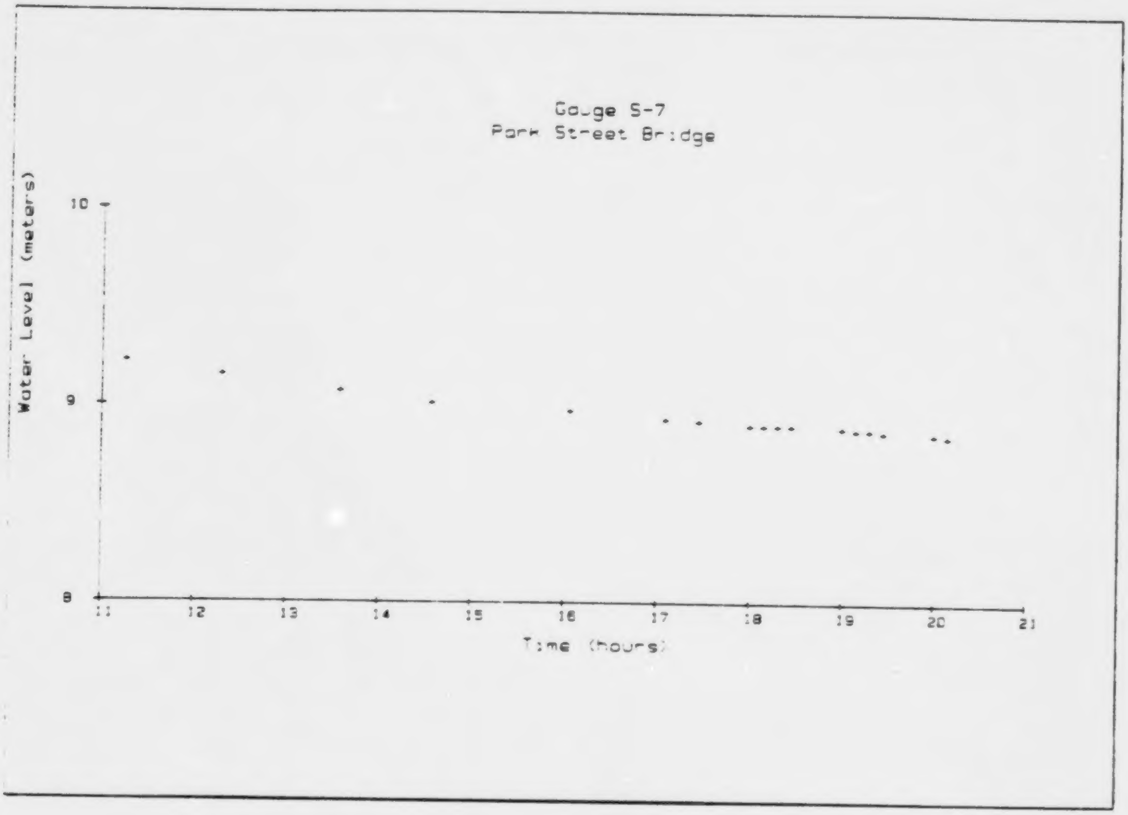
Hour	Water Level
11.14	8.01
12.22	7.88
13.50	7.76
14.47	7.69
15.56	7.61
18.00	7.46
19.00	7.78
20.00	7.46

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS FARNHAM BROOK

APRIL 29, 1982

FIGURE A.9



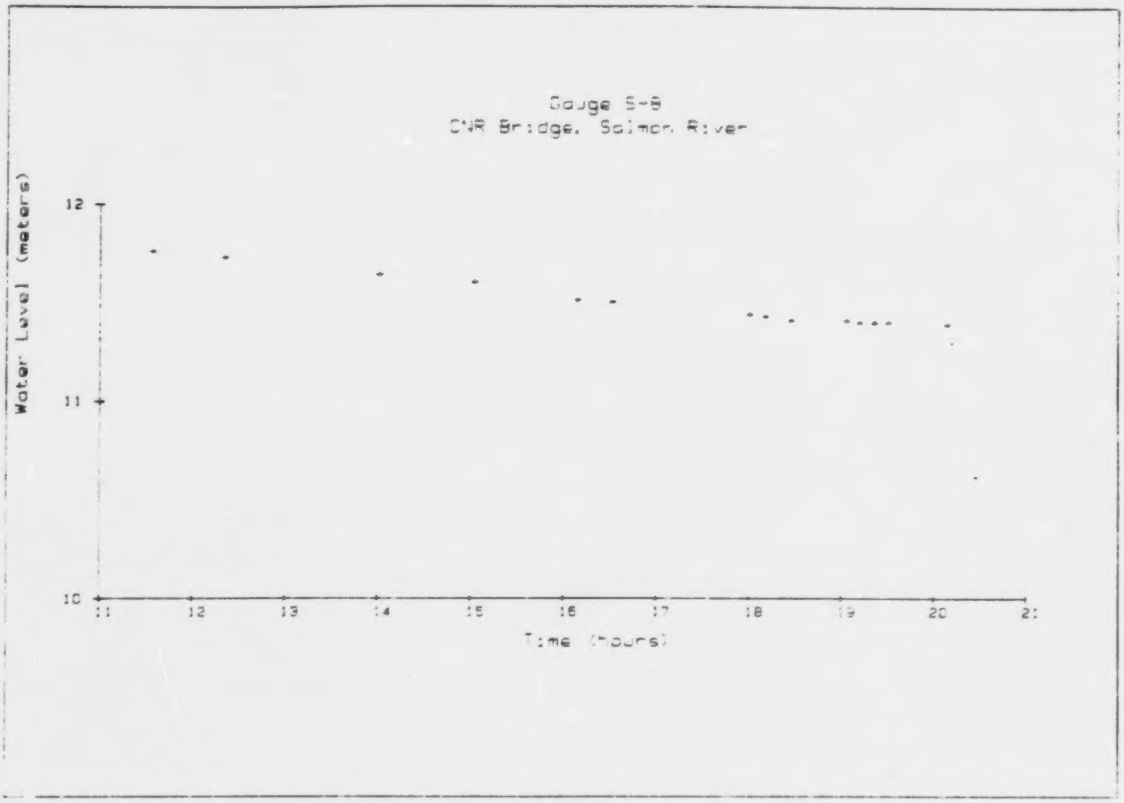
Hour	Water Level
11.26	9.22
12.30	9.15
13.58	9.07
14.57	9.01
16.06	8.97
17.09	8.93
17.45	8.92
18.00	8.90
18.15	8.90
18.30	8.90
18.45	8.90
19.00	8.89
19.15	8.88
19.30	8.88
19.45	8.87
20.00	8.86
20.15	8.85

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS PARK STREET BRIDGE

APRIL 29, 1982

FIGURE A.10

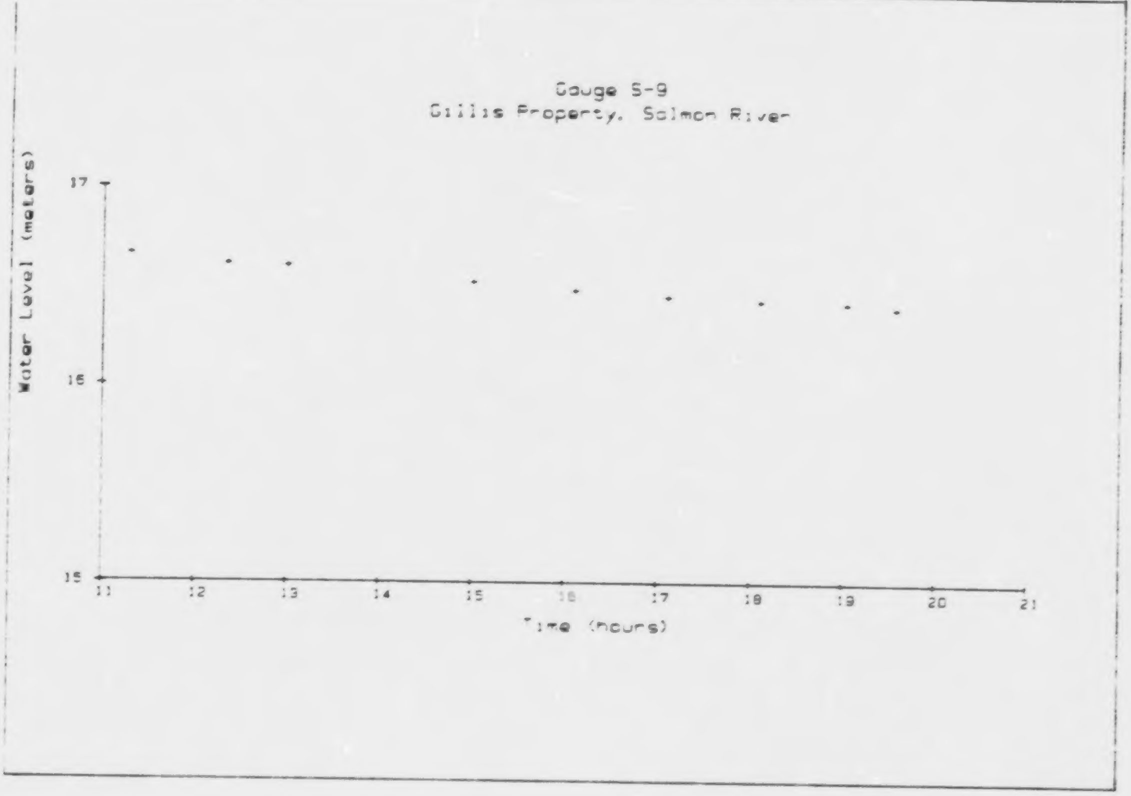


Hour	Water Level
11.57	11.76
12.35	11.73
14.02	11.64
15.05	11.60
16.16	11.51
16.53	11.50
18.01	11.44
18.18	11.43
18.46	11.41
19.06	11.41
19.20	11.40
19.36	11.40
19.51	11.40
20.15	11.39

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS C.N.R. BRIDGE
APRIL 29, 1982

FIGURE A.11



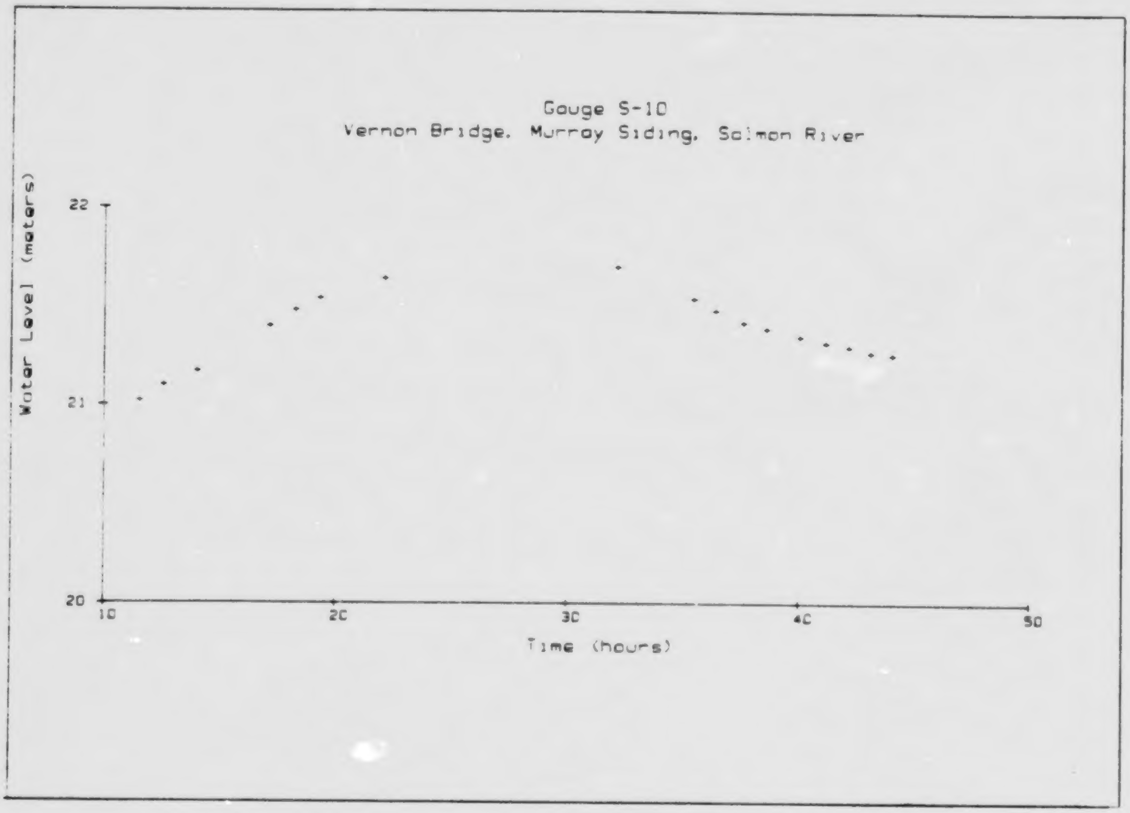
Hour	Water Level
11.30	16.66
12.35	16.61
13.00	16.60
15.00	16.52
16.10	16.48
17.10	16.45
18.10	16.43
19.03	16.42
19.57	16.40

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS GILLIS FARM

APRIL 29, 1982

FIGURE A.12



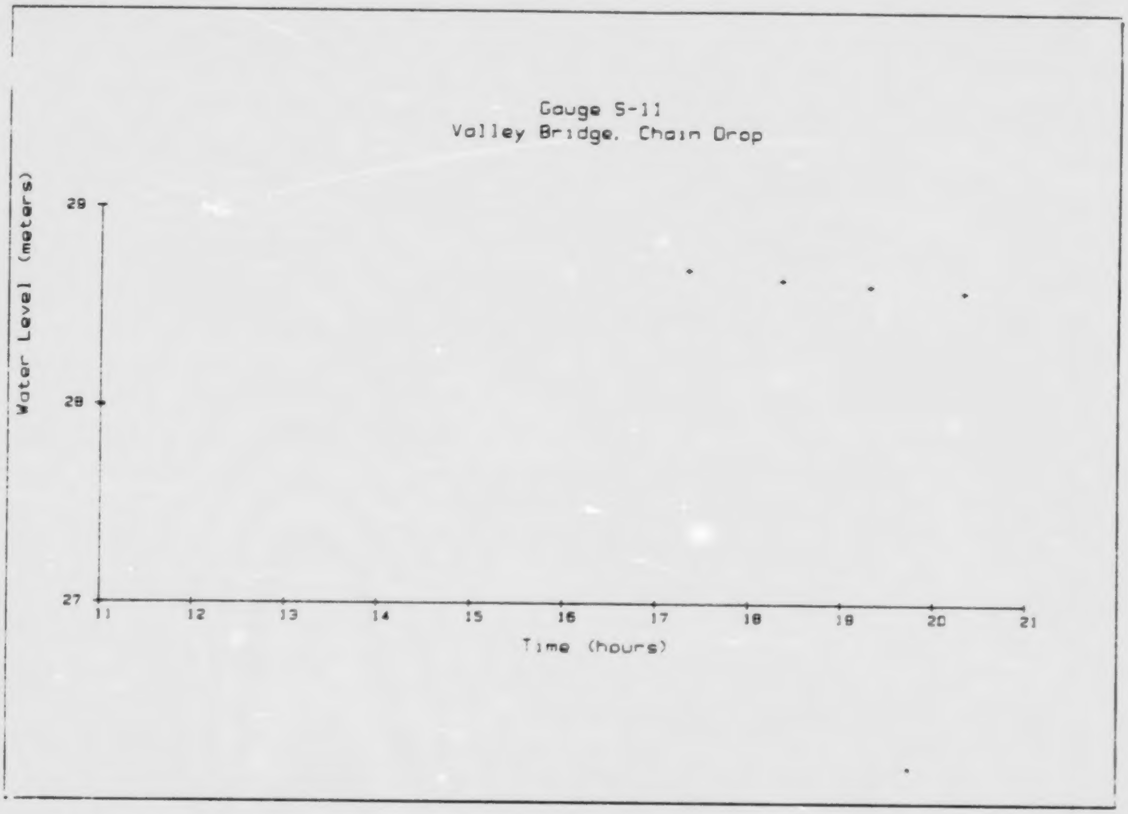
	Hour	Water Level
APRIL 28	11.56	21.02
	12.59	21.10
	14.02	21.17
	17.15	21.40
	18.25	21.46
	19.30	21.54
	22.10	21.64
APRIL 29	32.14	21.70
	35.44	21.54
	36.38	21.48
	37.59	21.42
	38.59	21.39
	40.03	21.35
	41.16	21.32
	42.16	21.30
	43.09	21.27
	44.03	21.26

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS, MURRAY SIDING BRIDGE

APRIL 29, 1982

FIGURE A.13

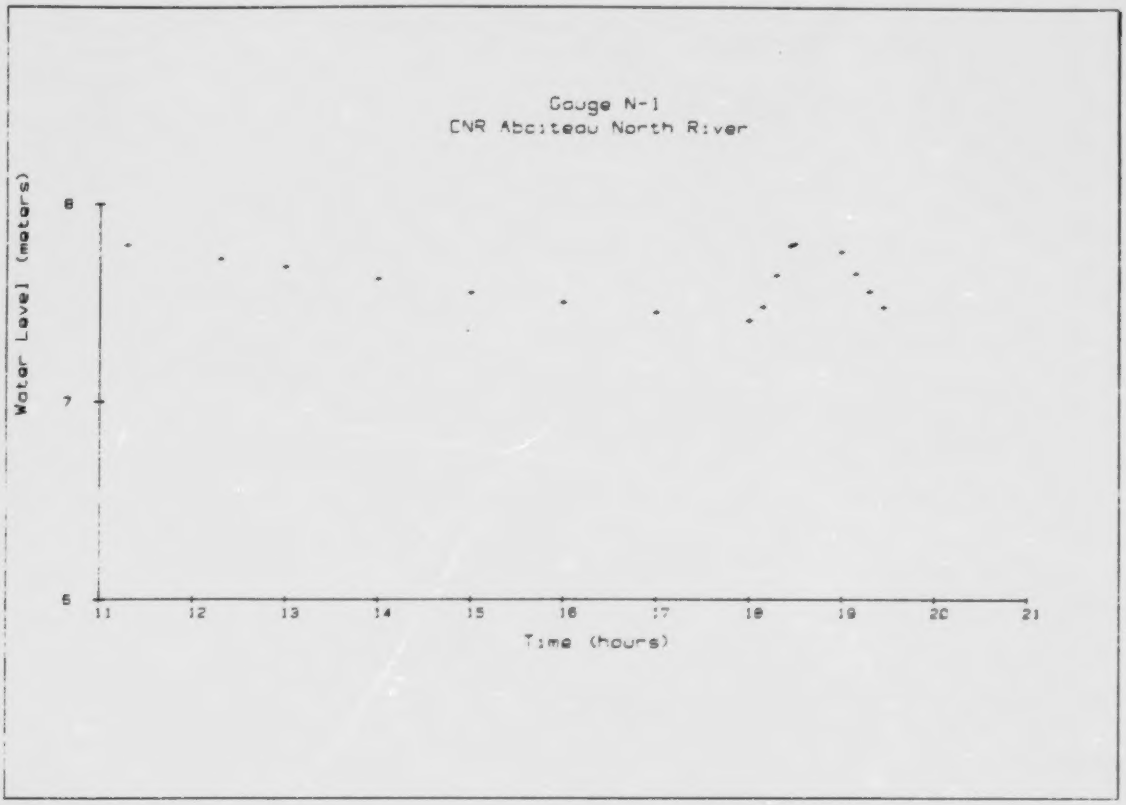


Hour	Water Level
17.35	28.69
18.36	28.64
19.31	28.61
20.32	28.58

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS VALLEY BRIDGE
APRIL 29, 1982

FIGURE A.14



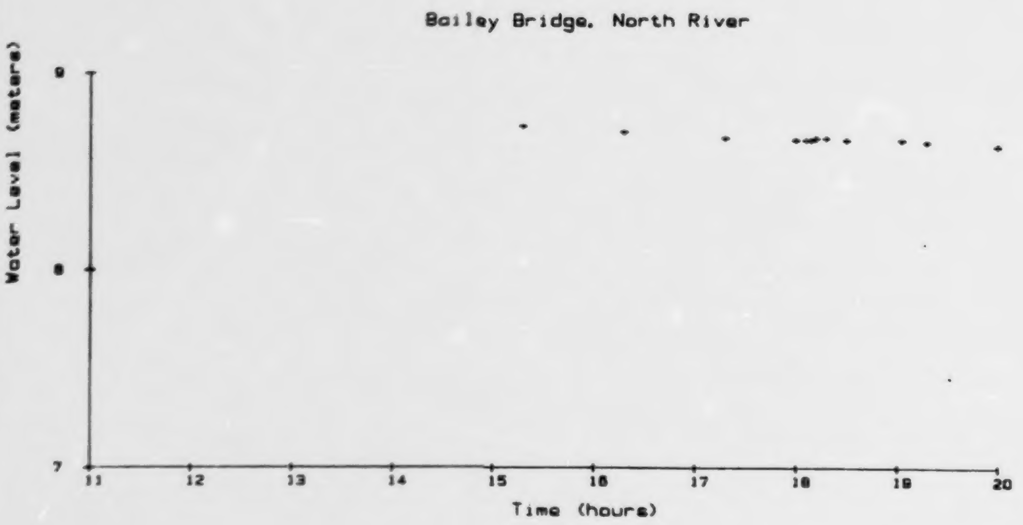
Hour	Water Level
11.30	7.79
12.30	7.72
13.00	7.68
14.00	7.62
15.00	7.55
16.00	7.50
17.00	7.45
18.00	7.41
18.15	7.48
18.30	7.64
18.45	7.79
18.50	7.80
19.00	7.76
19.15	7.65
19.30	7.56
19.45	7.48

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS, C.N.R. ABOITEAU, NORTH RIVER

APRIL 29, 1982

FIGURE A.15



Hour	Water Level
15.30	8.73
16.30	8.70
17.30	8.67
18.00	8.66
18.10	8.66
18.15	8.66
18.20	8.67
18.30	8.67
18.50	8.66
19.05	8.66
19.30	8.65
20.00	8.63

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM

TRURO HYDROTECHNICAL STUDY

WATER SURFACE ELEVATIONS BAILEY BRIDGE, NORTH RIVER

APRIL 29, 1982

FIGURE A.16

APPENDIX B

BACKGROUND TO HYDROLOGY

APPENDIX B

BACKGROUND TO HYDROLOGY

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B.1 REGIONAL DESIGN FLOW ESTIMATES

B.1.1 Peak Flow Estimates

The regional prediction equation for daily mean flood peak was found to take the following general form:

$$\ln QD_T = K + a \ln A + b \ln MAR + C \ln MSM + d \ln LONG + e \ln DF \qquad (B.1)$$

where QD_T = mean daily peak flow for recurrence interval T (T=1:2, 1:20, 1:100 years)

K,a,b,c,d,e = regression coefficients

A = drainage basin area (km^2)

MAR = index of mean annual runoff (mm)

MSM = index of median depth of snow on the ground at the end of March (mm)

LONG = degrees longitude of the basin outlet (nearest 5 seconds)

DF = drainage density factor, which is the number of permanent stream confluences divided by drainage basin area

The equation coefficients are summarized in Table B-1. The specific basin parameters and results of application are summarized in Table B-4. The specific basin parameters were found to be within the range prescribed by Tables B-2 and B-3.

B.1.2 Sensitivity Analysis

Sensitivity analysis was carried out to show the effect of parameter changes in the regression equation on the computed values of QD and QP. This was done by varying one parameter while holding the others at their initially determined values. The summary contained in Tables B-5 and B-6 shows the percent variation in peak flow estimates for the indicated parameter variation. It is seen

TABLE B.1

Regression Equations for Daily Mean Flood Peaks

Daily Mean Flood Peak: $\ln QD = K + a \ln A + b \ln MAR + c \ln MSM + d \ln LONG + e \ln DF$

Recurrence Interval (years)	K	a	b	c	d	e	R^2	SE of Estimate Log Units	SE of Estimate %	± 95% Confidence Limits %
2	65.167	0.916	-1.108	0	-14.032	.191	0.98	0.16	+17, -15	+37, -27
20	44.993	0.849	0	.152	-10.925	.140	0.95	0.23	+26, -21	+57, -36
100	43.367	0.836	0	.183	-10.480	.127	0.93	0.27	+31, -24	+70, -41

NOTE: Above equations were developed using natural logarithmic transforms of the variables.

R^2 = coefficient of multiple determination of the logarithmic values

SE = relative standard error of the regression equation

TABLE B.2

Range of Independent Variables Used in Developing the Regression Equations for QD_2, QD_{20}, QD_{100}

Parameter	Range
A $(km)^2$	26.9 - 1450.0
DF (confluence/km^2)	0.06 - 0.24
MAR (mm)	770 - 1000
MSM (mm)	15 - 200
LONG (degrees)	61.46 - 66.12

TABLE B.3
Range of Independent Variables Used in Developing the Regression Equations for QP/QD

Parameter	Range
A $(km)^2$	26.9 - 1390
MAR (mm)	780 - 1000
LONG (degrees)	61.46 - 66.12
MSM (mm)	15 - 200

TABLE B.4

Regional Flow Estimates

Stream Location	Drainage Area km^2	MAR mm	MSM MM	LONG Deg.	DF $\frac{1}{km^2}$	Daily Mean Flood Flow (QD m^3/s)		QP/QD	Instantaneous Flood Flow (QP m^3/s)			
						20 yr	100 yr		20 yr	95% C.L.	100 yr	95% C.L.
Salmon River at Valley	316	800	130	63.197	0.098	148.4	205.5	1.434	212.8	+70 -41	294.7	+82 -45
North River at North River	201	800	80	63.256	0.084	90.6	124.6	1.56	141.3	+82 -45	194.4	+94 -48

that the flow estimates are most sensitive to changes in the watershed area and the index values of MAR and LONG. Generally, the index of longitude (LONG) can be derived from existing mapping or surveys without error.Chances of error for LONG as high as those indicated in Tables B.5 and B.6 may not occur. The other two parameters, A and MAR, are determined most accurately from the available information and variations of up to 10 percent are not likely.

It is evident that much larger variations in the estimates of the MSM and DF indices are required before variations in the flow estimates similar to variations in A and MAR parameters may occur. The results indicate that variations in the peak flow estimates are within reasonable limits when the parameter values are within the range on which the regional analysis was based. (For example, when MAR is at -10% of the initial value it lies outside the range of application of the equation and the variation in peak estimates increases substantially.) However, all variations are well within the expected statistical confidence limits.

B.2 Design Flood Hydrographs for Regional Flows

B.2.1 Timing of Peak Flows

Peak open water flood stages occur due to the combined effect of freshwater flood flows and tides and their dissipation through large storage areas along the Salmon River estuary. Therefore, timing of design flood flows in the form of a design flood hydrograph was required for routing of inflows from the Salmon River, North River and their tributaries.

The time of occurrence of peak flows was analysed by examination of recorded flow hydrographs at various locations on the two rivers. A comparison of timing of peaks on the two rivers was obtained by examining the actual hourly records for selected flood events from the period 1977 to 1982 as summarized in Table B-7. It was found that there is an apparent phase difference varying from 1 to 2

TABLE B.5

Sensitivity Analysis Salmon River at Valley

			20 Year Flows						100 Year Flows			
Parameter	% Change	Value	QD_{20}	% Vari	QP_{20}	% Vari	$\frac{QP}{QD}$	% Vari	QD_{100}	% Vari	QP_{100}	% Vari
A	+5	331.8	154.62	+4	220.18	+3	1.42	-1	213.27	+4	303.70	+3
	-5	300.2	142.02	-4	205.07	-4	1.44	+1	196.16	-5	283.25	-4
	+10	347.6	160.77	+8	227.65	+7	1.42	-1	221.73	+8	313.97	+7
	-10	284.4	135.61	-8	197.04	-7	1.45	+1	187.49	-8	272.42	-7
MAR	+5	840	N/A	-	194.25	-9	1.31	-9	N/A	-	269.10	-9
	-5	760	N/A	-	240.36	+13	1.62	+13	N/A	-	332.84	+13
	+10	880	N/A	-	181.70	-15	1.22	-15	N/A	-	251.61	-15
	-10	720	N/A	-	282.15	+32	1.90	+32	N/A	-	390.71	+32
MSM	+10	143	150.04	+1	214.0	+1	1.43	0	208.35	+1	297.11	+1
	-10	117	145.54	-2	210.0	-1	1.44	+1	200.83	-2	289.11	-1
	+20	156	152.04	+3	215.7	+1	1.42	-1	211.69	+3	300.40	+2
	-20	104	142.95	-3	207.6	-2	1.45	+1	196.56	-4	285.50	-3
DF	+10	.108	149.91	+1	214.9	+1	N/A	-	207.29	+1	297.26	+1
	-10	.088	145.67	-2	208.9	-2	N/A	-	201.97	-2	289.63	-2
	+30	.127	153.35	+3	219.9	+3	N/A	-	211.60	+3	303.44	+3
	-30	.069	140.79	-5	201.9	-5	N/A	-	195.83	-5	280.82	-5
LONG	+.003	63.39	143.11	-3	197.41	-7	1.38	-4	198.41	-3	293.68	-7
	-.003	63.01	154.83	+3	228.74	+8	1.50	+4	211.31	+3	316.26	+7
	+.005	63.51	140.04	-5	188.65	-11	1.35	-6	194.32	-5	261.76	-11
	-.005	62.88	156.21	+5	241.07	+13	1.54	+7	215.79	+5	333.03	+13

TABLE B.6

Sensitivity Analysis North River at Stewart Bridge

			20 Year Flows						100 Year Flows			
Parameter	% Change	Value	QD_{20}	% Vari	QP_{20}	% Vari	$\frac{QP}{QD}$	% Vari	QD_{100}	% Vari	QP_{100}	% Vari
A	+5	211.05	94.46	+4	146.13	+3	1.55	-1	129.82	+4	200.83	+3
	-5	190.95	86.76	-4	136.45	-3	1.57	+1	119.40	-4	187.82	-3
	+10	221.10	98.26	+8	150.93	+7	1.54	-1	134.96	+8	207.30	+7
	-10	180.90	82.87	-8	131.52	-7	1.59	+2	114.12	-8	181.11	-7
MAR	+5	840	N/A	-	126.75	-10	1.40	-10	N/A	-	174.31	-10
	-5	760	N/A	-	162.95	+15	1.80	+15	N/A	-	224.03	+15
	+10	880	N/A	-	116.78	-17	1.29	-17	N/A	-	160.61	-17
	-10	720	N/A	-	195.83	+38	2.16	+38	N/A	-	269.26	+38
MSM	+10	88	91.95	+1	142.52	+1	1.55	-1	126.82	+2	196.57	+1
	-10	72	89.19	-2	140.12	-1	1.57	+1	122.25	-2	192.05	-1
	+20	96	93.17	+2	143.57	+2	1.54	-1	128.86	+3	198.57	+2
	-20	64	87.61	-3	138.68	-2	1.55	+1	119.64	-4	189.39	-3
DF	+10	.092	91.79	+1	143.16	+1	N/A	-	126.08	+1	196.65	+1
	-10	.076	89.37	-1	139.39	-1	N/A	-	123.05	-1	191.92	-1
	+30	.109	94.00	+4	146.61	+4	N/A	-	128.82	+3	200.92	+3
	-30	.059	86.25	-5	134.53	-5	N/A	-	119.16	-4	185.85	-4
LONG	+.003	63.45	83.17	-3	126.86	-7	1.49	-4	120.77	-3	179.86	-7
	-.003	63.07	88.82	+3	145.68	+8	1.64	+5	128.62	+3	210.98	+8
	+.005	63.57	81.39	-5	117.82	-12	1.45	-7	118.28	-5	171.23	-12
	-.005	62.94	90.78	+5	154.35	+15	1.77	+9	131.34	+5	223.33	+14

TABLE B.7
Timing of Peak Flows* for North River and Salmon River

Date	North River at North River Peak Time	Salmon River Peak Time	Difference Hours
April 29, 1982	1 a.m.	1 a.m. (Valley)	0
April 6, 1981	6 p.m.	8 p.m. (Union)	2
April 5, 1980	11 a.m.	10 a.m. (Union)	1
Nov. 14, 1979	8 p.m.	9 p.m. (Union)	1
March 15, 1979	9 a.m.	11 a.m. (Union)	2
Sept. 3, 1977	3 a.m.	3 a.m. (Murray)	0

* Timing of peak flows is to the nearest hour as determined from recorded hourly discharge values.

hours between North River at Stewart Bridge and Salmon River at Union. However, it was found that the Murray and Valley gauges could be considered to peak at the same time as the North River. These stations are 5 to 7 km from the confluence of the two rivers whereas Union gauge is about 13 km upstream of the confluence. Therefore, considering the distance to the Union gauge, and a phase difference compared to the North River of only 1 to 2 hours, it is reasonable to assume that both rivers peak at the same time at the location of the upstream limits of the study area.

B.2.2 Shape of Design Hydrographs

The shape of the 1:20 and 1:100 year design flood hydrographs for the Salmon and North Rivers was determined based on recorded hydrograph shapes. Ratios of the hourly discharges to the peak discharges from the Salmon and North River records for several events are plotted on Figures B-1 and B-2. These ratio plots were averaged and the mean ratios are also shown in Figures B-1 and B-2. The 1:20 and 1:100 year instantaneous peak flows estimated from the regression equations were then multiplied by these mean ratios and thus 1:20 and 1:100 year design flood hydrographs were obtained for both rivers as shown in Figures 6-3 and 6-4 of the main report.

B.3 HYMO Background

B.3.1 Model Calibration

The HYMO model was calibrated for the storm of August 1971 (Hurricane Beth). The Curve Number (CN) was estimated for the Fraser Brook watershed by analysing the overall effect of soil types, vegetation cover and land use characteristics. Information concerning soil characteristics such as slope, stoniness, drainage, rockiness, soil profile, soil association name and soil group, was obtained from soil maps and other information supplied by the Soil and Crop Branch of the Nova Scotia Department of Agriculture and Marketing. This information is summarized in Table B-8.

TABLE B.8

Estimation of Runoff Curve Number for Fraser Brook

No.	Soil Classification (N.S.D.A. Soil Maps)	Area (km^2)	Soil Characteristics	Hydrologic Soil Group	Land Use	Group CN (II)
1.	i) HO^4/d2	1.29	Dominantly imperfect drained, gravelly loam, 5.1 to 9% slope, moderately stony 0.1 to 3%	C	100% wood forest	77
	ii) HO^6/c3	0.57	Significantly poorly drained, gravelly loam, 2.1 to 5% slope, very stony 3 to 15%	C	100% wood forest	77
2.	i) HO^3/d3	2.37	Moderately well drained, gravelly sandy loam, 5.1 to 9% slope, very stony 3 to 15%	BC	98% wood forest 2% open spaces	63 74
	ii) He^3/d2	0.31	Moderately well drained, gravelly loam, 5.1 to 9% slope, moderately stony 0.1 to 3%	BC	98% wood forest 2% open spaces	63 74
	iii) HO^3/c3	4.53	Moderately well drained, gravelly sandy loam 2.1 to 5% slope, very stony 3 to 15%	BC	98% wood forest 2% open spaces	63 74

Total area = 9.07 km^2 Weighted CN (Condition AMC II) = 66.0

The curve number directly affects the potential volume of runoff during a storm and can, therefore, be used to describe various antecedent soil moisture conditions. Condition II is the average case for floods. On the other hand, the AMC III condition is related to a situation when heavy rainfall results in saturated soil or when the ground may be frozen during the five days previous to the given storm. An analysis of the hydrologic soil groups and land use in the Fraser Brook resulted in an estimated curve number of 66 (Table B.8). The CN value was checked using the actual antecedent conditions prior to the August 15, 1971 storm (Hurricane Beth) based on the baseflow, runoff and precipitation. The average baseflow at the start of the recorded hydrograph for the first three hours was 0.08 m^3/s, representing the average conditions. The recorded runoff volume was 57.8 mm and the precipitation was 139 mm. The equation given in Section B.3.3 for computing direct runoff volume was used to estimate CN. With the known values of Q and P, S was computed and application of the second equation gave a CN value of 69 which is close to the estimated value and was selected as the calibrated value for Fraser Brook. For saturated conditions (AMC III) the value of CN is increased by 20 percent to 82.8 according to curves given in SCS Handbook (SCS, 1972).

The other two input parameters for the HYMO model; the recession factor (k) and the time of peak (t_p) of the unit hydrograph were initially based on the empirical relationships given in the HYMO users manual (U.S. Dept. Agr., 1973). The estimated initial values of $k = 1.0$ and $t_p = 1.0$ hours were used in the HYMO calibration runs. Several runs were then made by varying k and t_p (at the same time fixing CN=69) until the recorded peak flow and runoff volume during the August 1971 storm were calibrated. This resulted in estimates of $k = 4.75$ and $t_p = 4$ hours. The resulting model parameters are summarized in Table B.9.

In undertaking the calibration, it was noted that although modifying k and t_p affected the timing and magnitude of peak flows, the simulated runoff volume was significantly changed only by modifying CN.

TABLE B.9
Fraser Brook Calibration
August 1971 Hurricane Beth Data

Parameter	Recorded or Estimated	Calibration Result
Runoff volume (mm)	57.8	58.0
Peak discharge rate (m^3/s)	5.95	5.94
Recession constant k (hours)	1.0	4.75
Time to peak t_p (hours)*	1.0	4.0
Curve number (CN)	66.0	69.0

* t_p is the unit hydrograph time to peak

It was determined by application of the hydrodynamic model that flood levels in the study area are more sensitive to changes in discharge volume as opposed to changes in peak flows from the tributary streams. Therefore, it was concluded that both the calibrated and uncalibrated model would provide flow estimates suitable for the purposes of the investigation.

B.3.2 Model Verification

The Fraser Brook HYMO model was verified for the February 1971 event. The calibrated values of $k = 4.75$ and $t_p = 4$ hours as calibrated for the August 1971 event, were applied as input for verification runs.

During the February 1971 storm, out of a total of 56 mm of recorded precipitation, 15 mm fell as snowfall and 31 mm was recorded as rainfall. Maximum recorded temperatures for the Truro area at two stations (Fraser Brook IHD and Truro) indicate that temperature peaked on February 12, 1971 at 44°F. On February 13 and 14 the minimum temperatures were 35°F and 32°F and the maximum ranged from 51°F to 54°F at Fraser Brook and Truro stations respectively. This indicates that a fair amount of snowmelt occurred during and prior to the mixed rainfall and snowfall storm. In fact, it was found that the antecedent moisture conditions were saturated due to snowmelt.

The results of the HYMO verification run, assuming saturated conditions and using the Fraser Brook IHD Station precipitation data, gave a CN value of 88.7 for which the timing and peak flow values were close to the observed data (see Table B.10). However, the computed runoff volume was about 30% compared to the observed value. This is attributed to snowmelt due to continuous mild temperatures for three days which likely contributed to a higher volume of runoff than represented by the rainfall input only. The HYMO model input and simulation results did not include this amount of snowmelt runoff.

TABLE B.10
Fraser Brook Model Validation
IHD February 13-14, 1971 Event

Parameter	Observed or Calibrated Value	Validation Run Results
Runoff volume (mm)	45.6	31.8
Peak discharge rate (m^3/s)	5.72	5.73
Recession constant k (hours)	4.75	4.75
Time to peak t_p (hours)*	4.0	4.0
Curve number (CN) (Saturated condition)	82.8	88.7

* t_p is the unit hydrograph time to peak

B.3.3 Soil Cover Complex Number (CN)

A runoff index factor combining the hydrologic soil group and land use characteristics is referred to as a hydrologic soil cover complex number, CN. Variations in antecedent soil moisture conditions can also be accounted for in each watershed by adjustment of the complex number. Simulated runoff volumes are proportional to the complex number according to the following rainfall-runoff relationship:

$$Q = \frac{(P - 0.2s)^2}{P + 0.8s}$$

where P = precipitation amount (inches)

$$s = \frac{1000}{CN} - 10$$

Q = amount of runoff (inches)

(Equation was originally developed in English units. To convert the amount of runoff to millimetres, multiply by 25.4).

Using the available land use and topographic mapping information, and soils information, as interpreted from surficial geology maps, the average soil cover complex number (AMC III) was determined for the watershed (refer to Tables B.11 and B.12 for typical soil cover complex numbers). The breakdown of land use characteristics and the resulting area weighted complex numbers are summarized in Table B.11, which also summarizes the percentage of forest cover, residential or urban land use, open area, recreational area, and industrial and commercial areas in each sub-watershed.

The complex numbers were determined for average and saturated soil conditions. The conversion relationship between AMC II and AMC III is summarized in Table B.13.

TABLE B.11

Watershed Hydrologic Characteristics

Name of Brook and Sub-basin	Hydrologic Soil Group	Land Use*	Weighted Curve Number	
			CN (II)	CN (III)
1. Christie Brook				
Sub-Area 1	80% BC	98F, 2P	69.0	82.8
(Fraser Brook)	20% C	100F		
Sub-Area 2	19% A	97F, 2P, 1U	67.0	81.0
	53% B	95F, 2U, 5R		
	28% C	100F, 2U, 3R		
2. Lepper Brook				
Sub-Area 1	1% B	100F	65.5	82.0
	27% C	91F, 2U, 7R		
	72% BC	95F, 5R		
Sub-Area 2	7% BC	100R	79.2	91.0
	7% A	75U, 12C, 13I		
	22% AB	44U, 5C, 7I, 44R		
	64% C	30U, 70R		
3. Farnham Brook				
Sub-Area 1	48% AB	100F	55.6	75.0
	52% C	100F		
Sub-Area 2	91% AB	80U, 10C, 10F	76.9	89.0
	5% C	80R, 20F		
	3% A	40U, 60F		
	1% B	100R		
Sub-Areas 3,4 & 5	15% A	100F	50.6	69.0
	27% C	100F		
	58% AB	100F		
4. McCurdy Brook				
Total Area	30% B	55P, 31F, 14U	71.0	85.0
	17% BC	60P, 39F, 1U		
	43% C	30P, 68F, 2U		
	10% D	25P, 75F		
5. McClures Brook				
Sub-Area 1	1% B	100F	79.9	86.0
	4% AB	65F, 10I, 25R		
	19% BC	56F, 38R, 6U		
	76% C	82F, 7R, 8U, 3I		
Sub-Area 2	29% C	85U, 6R, 9I	82.2	92.0
	71% AB	61U, 15R, 15I, 9C		
Sub-Area 3	41% A	76U, 6R, 18C	79.7	91.0
	59% AB	53U, 14R, 33C		
6. Soley Brook				
Sub-Area 1	27% B	98P, 2U	72.0	86.0
	73% C	41P, 59F		
Sub-Area 2	100% C	90P, 10F	77.0	89.0
7. Higgins Brook				
Total Area	32% B	75P, 10U, 15F	76.0	89.0
	68% C	70P, 30F		

Land Use

F - % Forestry
U - % Residential or Urban
P - % Open area
R - % Recreational
I - % Industrial
C - % Commercial

TABLE B.12
Summary of Typical Runoff Curve Numbers

Land Use	Hydrologic Soil Group* B	BC	C	CD	D
Residential	85	88	90	91	92
Row Crop	71	74	78	79	81
Forest	66	71	77	80	83
Pasture	69	74	79	82	84

* Based on Antecedent Moisture Condition II

Source: (S.C.S., 1972)

TABLE B.13

Relationship Between AMC II and AMC III

CN for AMC II	Corresponding CN for AMC III
100	100
95	98
90	96
85	94
80	91
75	88
70	85
65	82
60	78
55	74
50	70
45	65
40	60
35	55
30	50
25	43
20	37
15	30
10	22
5	13

AMC II	The average condition for antecedent soil moisture
AMC III	Highest runoff potential. Soils in the watershed are saturated from antecedent rains

Source: (S.C.S., 1972)

B.3.4 Design Storms

i) Storm Volume

It was necessary to input the 1:20 and 1:100 year design storms in order to estimate the corresponding peak flows. Two meteorological stations, Truro and Fraser Brook IHD (Figure 6-8) are located in the study area. The meteorological record of Fraser Brook is short and is available only for the period 1966-1975. Truro, on the other hand, has long-term records for the periods 1870-1915 and 1958 to date. For design storm analysis only Truro data for the period of record 1958 to 1977 were used as they are recent and continuous. The record period chosen also reflects the time of commencement of the hydrologic study. Although the longer period meteorological record (1958-1984) was later obtained, the differences between the two sets of data were considered minor. Table B.14 contains a summary of the rainfall-intensity-duration-frequency data for Truro.

The confidence limits for rainfall totals for any recurrence interval can be determined using the following relationship:

$$CL_{95} = 1.96\ S_e = 1.96\ (1 + 1.14\ k + 1.10\ k^2)^{1/2} \frac{S_x}{\sqrt{n}}$$

$$= 1.96\ (\delta) \frac{S_x}{\sqrt{n}}$$

where CL_{95} = 95% confidence limits

S_e = standard error of estimate

S_x = standard deviation (the value is 17.4 mm based on 24 hour duration storm for 19 years of record

n = number of years of record = 19

k = frequency factor

δ = a constant dependent on k and sample size n obtained from Table 8.4, page 176, "Frequency and Risk Analysis in Hydrology", G. W. Kite (1976)

TABLE B.14

Rainfall at Truro (mm)*

Return Period Duration	1:2 yrs	1:5 yrs	1:10 yrs	1:25 yrs	1:50 yrs	1:100 yrs
5 min	5.4	7.3	8.5	10.1	11.3	12.5
10 min	8.4	11.5	13.5	16.0	17.9	19.7
15 min	10.5	13.9	16.2	19.1	21.3	23.4
30 min	14.3	19.9	23.7	28.4	31.9	35.4
60 min	19.2	26.7	31.6	37.9	42.6	47.2
2 hrs	26.7	37.4	44.5	53.5	60.1	66.7
6 hrs	43.0	55.5	63.8	74.2	82.0	89.7
12 hrs	50.5	63.3	71.8	82.6	90.6	98.5
24 hrs	59.4	78.0	90.4	105.9	117.5	129.0

* total rainfall for indicated recurrence interval and duration (mm)

For example, the upper 95% confidence limit was computed for the 1:20 and 1:100 year 24-hour amounts and is summarized as follows:

Parameter	1:20 year	1:100 year
n	19	19
S_x	17.4	17.4
δ	3.10	4.67
CL_{95} (L) mm	24.25	36.53
24-hr storm (I) mm	102.7	129.00
L + I (mm)	126.95	165.53

ii) Critical Storm Duration

Rainfall storms of 6, 12, 18 and 24-hour duration were considered for comparison. The HYMO model was applied to Fraser Brook with variable Mass rainfall values for different durations. Runoff volume and peak flows were computed for various storm durations as given in Table B.15.

A comparison of tabulated values indicates peak flows do not vary much with change in duration. However, there is upward trend in runoff volume, being lower for 6-hour storm and maximum for 24-hour storm. In the application of the hydrodynamic model, volume of runoff was found to be an important and influencing factor. Therefore, the 24-hour duration storm was selected for this study as this gave the highest volume and would result in more conservative flood levels.

Previous investigators have also used this Truro precipitation data and period of record from 1958 to 1977 (Dillon, Horner, 1981). It was found that the rainfall distribution for one hour and 12-hour storms could be represented on a dimensionless duration scale, shown as 10, 50 and 90 percent curves on Figure B-3, and these curves can be used for any storm duration in the Truro area. The 10, 50 and 90 percent curves on Figure B-3 indicate variations in rainfall patterns for the corresponding percentage of real events

TABLE B.15

Fraser Brook Storm Duration Analysis

20-year Design Storm

Storm Duration (hrs)	CN (II) Peak Flow (m^3/s)	CN (II) Runoff Volume (mm)	CN (III) Peak Flow (m^3/s)	CN (III) Runoff Volume (mm)
6	3.87	14.5	8.50	32.4
12	4.30	18.6	8.42	38.5
18	4.99	27.0	8.72	50.6
24	4.95	32.6	8.24	58.2

analysed. For example, the use of the 10 percent curve would result in an advanced pattern of excess rainfall and the 90 percent curve would provide a delayed pattern resulting in high rainfall values toward the end of the storm.

The 50 percent storm distribution curve representing the average rainfall excess conditions (Figure B-3) was chosen for the 1:100 year storm distributions over a 24-hour period. The corresponding mass rainfall accumulated at 30 minute intervals is given in Table 6.7 for both the mean total volume and the upper 95% confidence limit values.

B.4 HYMO Model Application to Salmon and North Rivers

The HYMO model was applied to the Salmon and North River watersheds in order to obtain secondary peak flow estimates for comparison to the estimates by the regional method. The Salmon River watershed was divided into 7 sub-areas in order to obtain peak flows at the study area boundary. Four distinct sub-areas were identified for South North River watershed whereas the West North River watershed was simulated as a single unit.

The physiographic parameters for the model application, such as length of stream, elevation difference, drainage area, channel and floodplain slopes and data for valley cross-sections, was based on the available 1:50,000 scale National Topographic series maps. The runoff complex number (for CN (I) and CN (II) conditions) was based on soil classification and land use information obtained from the Soil Map of Colchester County prepared by the Federal Agriculture Department on the basis of soil surveys done by the Nova Scotia Department of Agriculture. The 24-hour design storm discussed in Section 6.4.5 was used as mass rainfall input. The model input parameters which were derived are considered to be adequate for the purposes of this comparison.

A summary comparison of the peak flows generated by the HYMO model and the Regional Analysis is given in Table B.16. The comparison confirms the results of the flow estimates by the Regional Analysis. The time to peak values obtained from the HYMO analysis are similar to those from the analysis of recorded hydrographs. While it is evident that there is a larger variation in peak flows for CN (II) or CN (III) conditions, it is evident from Table B.16 (compared to Table 6.3) that the variations are well within the confidence limit estimates of the regional method. Furthermore, the average of the CN (II) and CN (III) peak flow estimates is quite close to the regional estimate. Since an intermediate "average" CN condition is quite possible (this could only be confirmed by detailed model calibrations which were considered to be outside the scope of the secondary comparisons), it was concluded that the peak flow estimates by the regional technique are reasonable and are suitable for the purposes of this investigation.

B.5 Fraser Brook Flood Frequency Analysis

Several distributions were fitted to the available Fraser Brook data set to obtain peak flow estimates for comparison purposes, including Gumbel, Log Normal (LN), Three Parameter Log Normal (3PLN) and Log Pearson Type III (LP3). The computer program, FDRPFFA was utilized for this purpose and for computing the 1:20 and 1:100 year flood peaks and the corresponding 95% confidence limits. The input data and associated statistics are summarized in Table B.17. Table B-18 presents a summary of the frequency analysis output for the different distributions and the corresponding standard errors.

In flood frequency analysis, it is not possible to pre-determine which frequency distribution should be used. The selection of the appropriate distribution for use in this investigation was based on comparisons of the computed "T" year flood using the methods as outlined in the FDRPFFA program (Env. Can., 1979), and summarized as follows:

- comparison of various fitting techniques to the regional peak flow estimates using available regional techniques
- comparison of the theoretical statistics of skew and kurtosis to those estimated from the station data
- visual comparison of the degree of fit of each distribution to the available data set
- estimation of upper limit of flow estimates.

For Nova Scotia, MacLaren (MacLaren Atlantic, 1980) recommends that the Three Parameter Log Normal Distribution be used.

TABLE B.16

Comparison of Regional Analysis and HYMO Model Results for Salmon and North Rivers

Name of Basin	Design Flow (1:Yr)	Regional Analysis Peak Discharge (m^3/s)	HYMO Model Peak Discharge (m^3/s)				Time to Peak (hrs)		
			Soil Condition		Variation from Regional Analysis		Recorded Hydrograph Shape Basis	HYMO	
			CN II	CN III	CN II	CN III		CN II	CN III
1. Salmon River	1:20	212.80	145.35	275.50	-32%	+29%	22.0	21.5	19.5
	1:100	294.70	230.44	384.88	-22%	+31%	22.0	21.0	19.0
2. North River									
South Branch	1:20	92.55	71.07	129.10	-23%	+39%	22.0	20.0	18.0
	1:100	127.33	111.31	179.75	-13%	+41%	22.0	19.5	17.5
West Branch	1:20	48.75	29.08	59.52	-40%	+22%	22.0	20.0	18.0
	1:100	67.07	47.63	84.30	-29%	+26%	22.0	20.0	18.0

TABLE B.17

Fraser Brook Flood Frequency Analysis

Date and Statistics

(Station 01HD003)

01HD003 Fraser Brook

Year	Month	Data	Ordered	Rank	Prob.	Return Period
1966	4	2.740	7.480	1	.043	23.026
1967	10	3.200	6.650	2	.101	9.943
1968	3	2.540	6.480	3	.158	6.341
1969	11	3.280	6.140	4	.215	4.654
1970	7	2.720	5.860	5	.272	3.676
1971	2	6.480	4.760	6	.329	3.038
1972	5	6.650	4.360	7	.386	2.589
1973	4	4.760	3.950	8	.443	2.255
1974	11	2.550	3.800	9	.501	1.998
1976	2	6.140	3.680	10	.558	1.793
1977	12	3.680	3.520	11	.615	1.626
1978	1	5.860	3.280	12	.672	1.488
1979	3	4.360	3.200	13	.729	1.371
1980	4	7.480	2.740	14	.786	1.272
1981	11	3.800	2.720	15	.843	1.186
1982	4	3.950	2.550	16	.901	1.110
1983	1	3.520	2.540	17	.958	1.044

Sample Statistics

Mean = 4. S.D. = 1.6 C.S. = 0.6772 C.K. = 2.7745

Sample Statistics (Logs)

Mean = 1.4052 S.D. = .3588 C.S. = .2971 C.K. = 2.4274
Sample Min = 3. Sample Max = 7. N = 17
Parameters for Gumbel 1 A = .852309 U = 4.
Parameters for Lognormal M = 1.4052 S = .3588

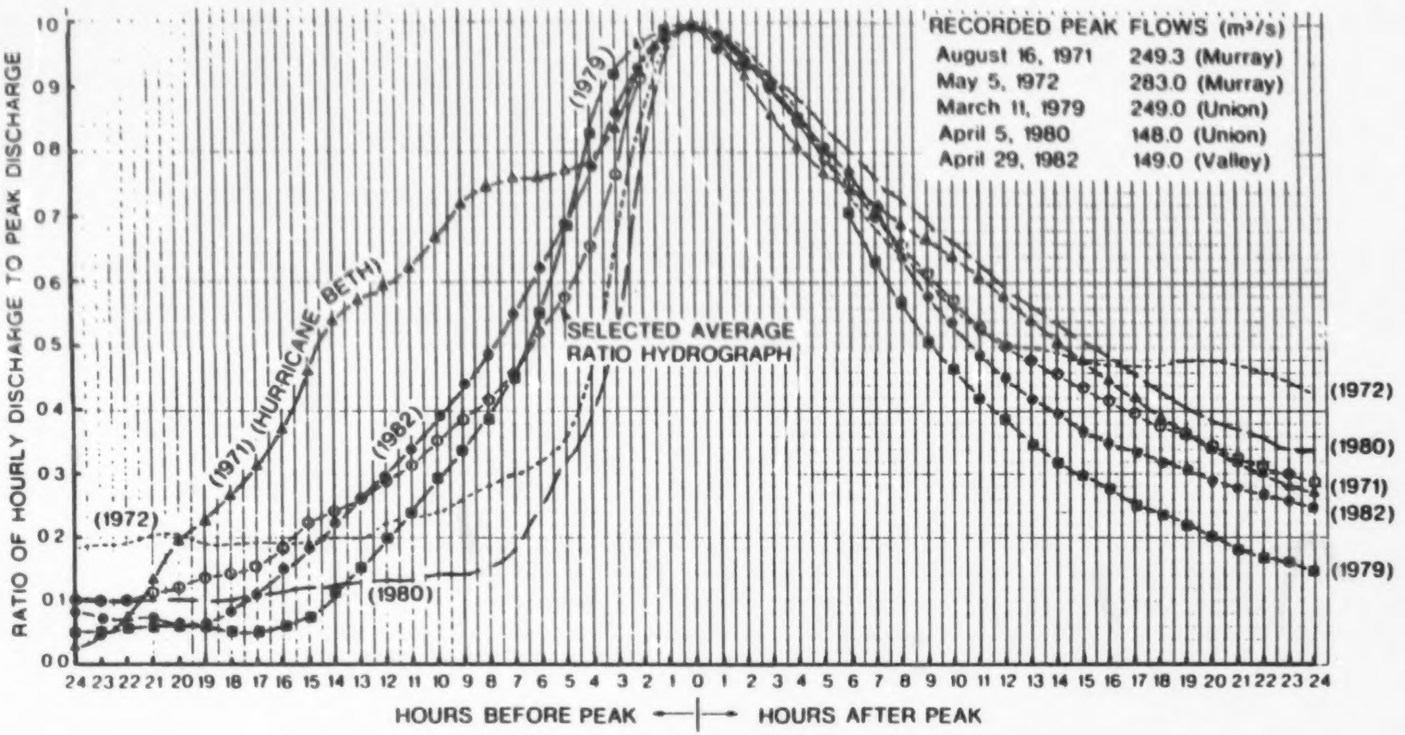
Statistics of Log (X-A)

Mean = .3752 S.D. = .9477 C.S. = -.4166 C.K. = 2.7464
Parameters for Three Parameter Lognormal A = 2. M = .3752 S = .9477
Parameters for Log Pearson III by Moments A = .0533 B = .4533E+02
Log(M) = -1.0105 M = .3640E+00
No maximum likelihood solution for Log Pearson III

TABLE B.18

Fraser Brook Flood Frequency Analysis
Peak Flows and Standard Error

	Gumbel I		Log Normal		Three Parameter Log Normal		Log Pearson III			
							Max. Likelihood		Moments	
Return Period	Flood Est.	Standard Error %	Flood Est.	Standard Error %	Flood Est.	Standard Error %	Flood Est.	Standard Error %	Flood Est.	Standard Error %
1.005	1.660	23.90	1.620	18.10	2.380	4.31	0.000	0.00	1.790	23.00
1.050	2.310	14.00	2.260	13.30	2.560	4.32	0.000	0.00	2.330	12.50
1.250	3.060	9.32	3.010	10.10	2.910	6.19	0.000	0.00	3.000	9.36
2.000	4.050	8.26	4.080	8.70	3.710	3.32	0.000	0.00	4.000	9.46
5.000	5.380	9.54	5.510	10.10	5.490	15.80	0.000	0.00	5.480	11.00
10.000	6.260	10.50	6.460	11.70	7.160	21.70	0.000	0.00	6.520	13.60
20.000	7.100	11.30	7.360	13.30	9.180	28.00	0.000	0.00	7.570	17.30
50.000	8.190	12.20	8.520	15.30	12.500	36.10	0.000	0.00	9.010	23.20
100.000	9.010	12.80	9.390	16.80	15.400	42.00	0.000	0.00	10.200	28.10
200.000	9.830	13.20	10.300	18.10	19.000	47.60	0.000	0.00	11.400	33.30
500.000	10.900	13.80	11.500	19.70	24.600	54.50	0.000	0.00	13.100	40.60

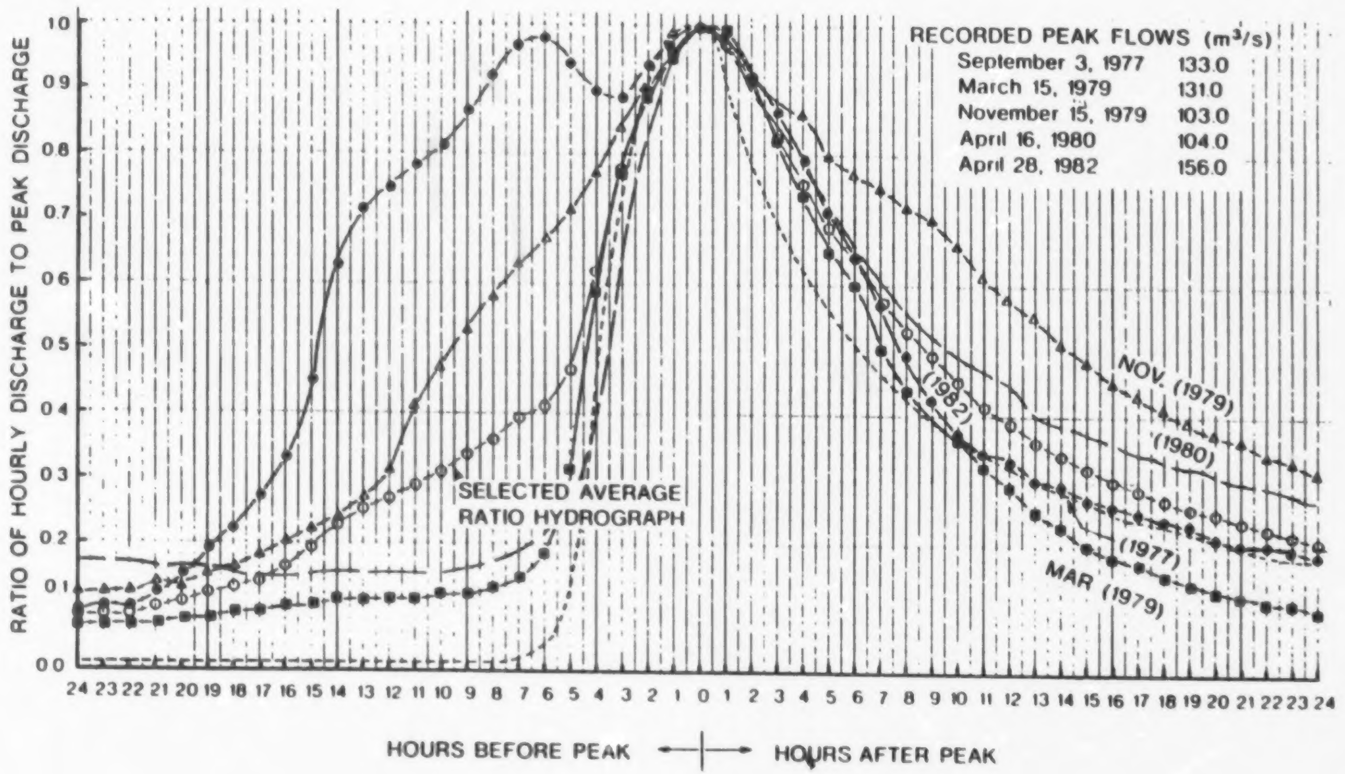


LEGEND

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

DIMENSIONLESS FLOOD FLOW HYDROGRAPHS FOR SALMON RIVER
(HISTORICAL FLOODS)

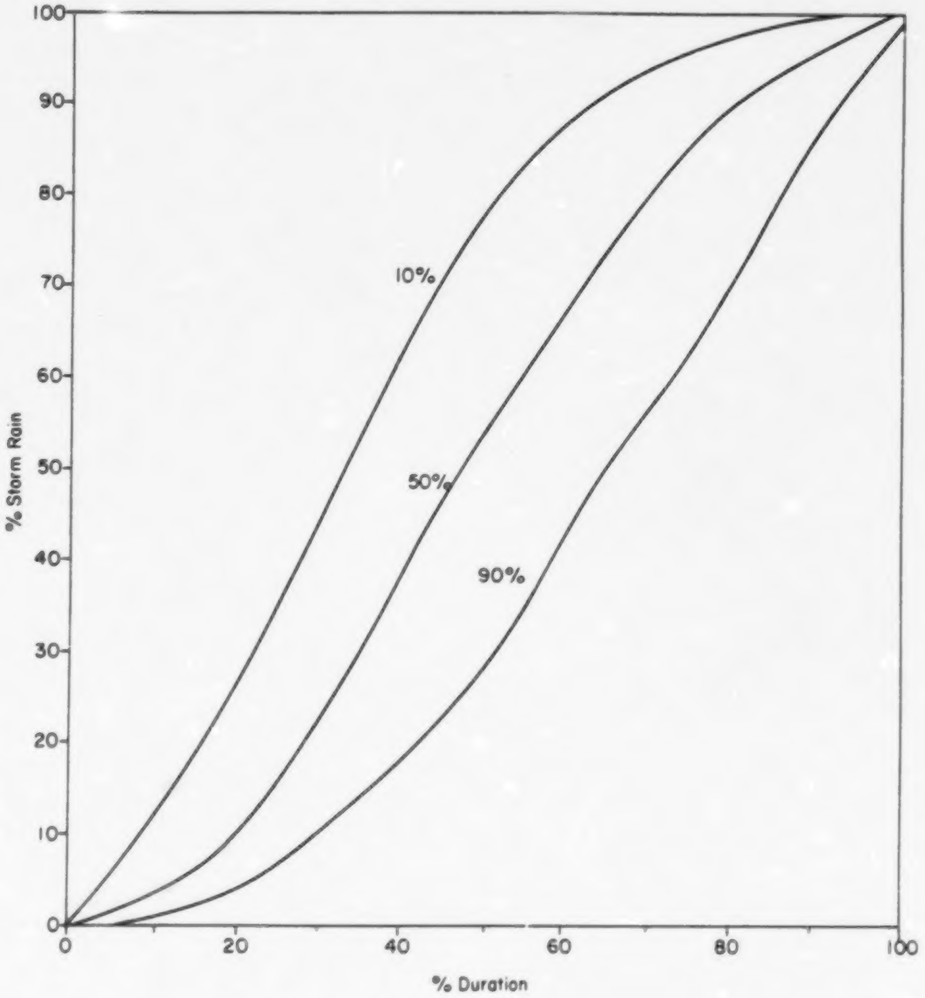
FIGURE B.I



CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

DIMENSIONLESS FLOOD FLOW HYDROGRAPHS FOR NORTH RIVER AT NORTH RIVER

FIGURE B.2



RAINFALL DISTRIBUTION AT TRURO

CANADA-NOVA SCOTIA
FLOOD DAMAGE REDUCTION PROGRAM
TRURO HYDROTECHNICAL STUDY

FIGURE B.3

APPENDIX C

BACKGROUND TO HYDRODYNAMIC MODELLING

APPENDIX C
BACKGROUND TO HYDRODYNAMIC MODELLING

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C.1 BACKGROUND TO HYDRODYNAMIC MODELLING

C.1.1 Introduction

Flows in both natural and man-made channels are usually unsteady. Normally in flood flow analysis, an assumption of steady state conditions for simplifying the computational procedures produces conservative results from a flood-stage point of view (Perks et al, 1983). The conservatively higher stage values arise from a steady state analysis because of the underlying assumption, that given a particular recurrence interval discharge, the flood wave is of infinite length thereby filling all channel storage and yielding no wedge storage. However, in certain flood studies, a steady-state analysis can lead to overly conservative results and even to erroneous low estimates in particular reaches of complex hydraulic networks. Examples of such studies would be: a dendritic river system where the flows from the tributaries cause backwater effects in the main channel or even reversals of flow; extensive embayment storage in channel reaches; and estuary reaches subject to tidal effects. Hydraulic computations for these cases require a more mathematically rigorous approach than for the traditional techniques such as the steady-state step method, channel storage routing, and Muskingum routing. The hydrodynamic modelling approach is recommended for these special cases.

The difficulty in formulating the solution of this problem increases with the degree of complexity of the general flow pattern (one, two or three dimensional). For flood study purposes, this discussion is limited to the solution of one-dimensional (1-D) flows.

C.1.2 Saint Venant Equations

Flood waves may generally be considered as 1-D gradually varied unsteady flows. The analytical foundation for the study of this type of flow is based on the work of Saint Venant, 1871. In order to obtain theoretical equations two principles are considered: the conservation of mass, or continuity (difference between inflow and outflow equals the time rate of change of storage) and the conservation of momentum (Newton's second law, stating that the sum of forces acting on a small element of water is equal to the net time rate of momentum leaving the element plus the time rate of accumulation of momentum in the element). Certain flow conditions must also be assumed:

a) the flow is 1-D (velocity is uniform over the cross-section and the water level across the section is assumed horizontal);
b) the streamline curvature is small, and vertical accelerations are negligible (pressure is hydrostatic);
c) the resistance equations developed for steady flows are applicable for unsteady flows (such as the Manning's or Chézy equations);
d) the average channel bed slope is small;
e) the flow is incompressible and its density is homogeneous;
f) the shear stresses due to wind and changes in atmosphere pressure are negligible.

Applying the conservation of mass and momentum laws to a non-presmatic channel (Figure C-1), the following two equatons are obtained:

Conservation of mass (continuity

$$\left.\underset{1}{B\frac{\partial z}{\partial t}} + \underset{2}{Bv\frac{\partial z}{\partial x}} + \underset{3}{A\frac{\partial v}{\partial x}} + \underset{4}{v\frac{\partial A}{\partial x}}\right| z = \text{const.} = \underset{5}{q_L} \qquad \text{(C-1)}$$

Conservation of momentum

$$\underset{6}{\frac{\partial v}{\partial t}} + \underset{7}{\frac{\partial v}{\partial x}} + \underset{8}{\frac{vq_L}{A}} = \underset{9}{g(S_o} - \underset{10}{\frac{v|v|}{C_z^2R}}\ ) - \underset{11}{g\frac{\partial h}{\partial x}} \qquad \text{(C-2)}$$

g = gravitational acceleration
h = depth of flow
q_L = lateral inflow per unit width
S_f = friction slope
S_o = bottom slope
t = time coordinate
v = mean flow velocity
v_x = velocity component of lateral inflow, in direction of the longitudinal axis
x = space coordinate, in direction of the longitudinal axis
z = water surface elevation
$(\partial A/\partial x)_z$ = variation of the cross-sections area in the direction of the longitudinal axis, for a constant water elevation, to account for non-prismatic characteristics of the sections.

Continuity Equation

The various terms contained in the continuity equation are defined below:

1) "Rate of rise" which gives the storage changes owing to water surface elevation changes with time.
2) "Prism storage" owing to variation in velocity with space.
3&4) "Wedge storage" owing to areal variations in velocity with space
5) "Lateral inflow" which gives the net mass change spatially and temporally beyond the storage terms.

Momentum Equation

The terms found in the momentum equation are given below:

6) Acceleration owing to time variation in flow
7) Acceleration owing to spatial variation in velocity
8) Acceleration effects owing to lateral inflow
9) Gravity body force owing to bed slope

10) Frictional force effect
11) Pressure force term.

For a wide rectangular channel the continuity equation takes the special form:

$$\frac{\partial h}{\partial t} + \frac{\partial \bar{b}}{\partial x} + \frac{\partial v}{\partial x} = q_L \qquad (C.3)$$

The preceding expressions differ slightly from the original Saint Venant equations because lateral inflow was not considered in the earliest formulation. The solution of these equations has become attainable for practical applications during the past two decades with the advent of high-speed computers and the accompanying advanced numerical methods.

The 1-D model is a FORTRAN coded computer program. Its modular structure is such that blocks (subroutines) can be easily added to the main program in order to extend its capabilities. The hydraulic features handled by the model are listed in the main text.

C.1.3 General Theory of 1-D Model

The governing equations (C-1 and C-2) can be rewritten in terms of discharge and Froude number. It can also be assumed that the lateral inflow enters the system with negligible momentum in the direction of the main flow, or in other words, with a small velocity component v_x. Expressing the friction slope in terms of the Chézy (or Manning) equation, the governing equations are written as:

Conservation of mass (continuity):

$$B \frac{\delta z}{\delta t} + \frac{\delta Q}{\delta x} = q_L \qquad \text{(C-4)}$$

Conservation of momentum:

$$\left[\frac{\delta Q}{\delta t} + 2v \frac{\delta Q}{\delta x}\right] \frac{1}{Ag} = (Fr^2-1) \frac{\delta z}{\delta x} + \frac{Fr^2}{B} \left(\frac{\delta A}{\delta x}\right)_z - \frac{Q|Q|}{K^2} \qquad \text{(C-5)}$$

where: C_z = Chézy's factor of flow resistance

$Fr = [v^2B/gA]^{1/2}$

$K = AC_z\sqrt{R} = AR^{2/3}/n$ (S.I.) or $1.486\ AR^{2/3}/n$ (Imperial)

n = Manning's coefficient of roughness

Q = discharge

R = hydraulic radius

In a finite difference technique, the continuous equations are approximated by discretized equations, also called finite differences. The hydraulic conditions are expressed in terms of time and space increments. For one time step at the time, the solution of the equations is found at each space increment in the system. The solution then proceeds to the next time step (Figure C-3).The time step is limited by the Courant-Levy-Friedrich criterion, or by some form of this criterion:

$$\Delta t \leq \frac{\Delta x}{v + |c|} \qquad \text{(C-6)}$$

where $|c|$ = absolute value of the celerity of the waves

Δt = time increment

v = mean flow velocity

Δx = space increment

This criterion is also known as the Courant criterion. In general terms, the observation of the Courant criterion can be understood as a limitation of the computational time period such that a wave will never go through a space increment without being considered in the computation process. The violation of the criterion would mean that a wave enters, goes across and leaves a space increment during the same period of time. The implicit schemes have the advantage of being unconditionally stable, with no restriction on the time step. However, in this case, other considerations such as the convergence of the solution may require some limitations on the computational time period, but those are less severe than the Courant criterion.

The finite difference scheme used in the 1-D model is an implicit scheme, and is thus unconditionally stable. It was developed at the M.I.T. by D. J. Gunaratnam and F.E. Perkins, 1970. The scheme was obtained by applying a weighted residual method of optimization to a simplified linearized version of the governing equations, in order to minimize the difference between the continuous equations and the discrete approximations (consistent scheme). Appropriate local and temporal adjustments were used in order to account for the non-linear characteristics of the governing equations. The convergence of the scheme, as defined from an engineering point of view, is found to be a complex function of the Froude number, the Courant condition, the friction and the waves characteristics. The following criteria were proposed for the convergence of the scheme:

$$\lambda/\Delta x \geq 100 \tag{C-7}$$

$$\Delta t \leq 5.5 \; \frac{\Delta x}{v + |c|} \tag{C-8}$$

where λ = wave length.

The condition expressed by equation C-6 shows that the scheme gives an accurate prediction even for the discretization of waves with relatively high frequency. A sensitivity analysis has demonstrated that the condition expressed by equation C-7 is very conservative for a situation predominated by low frequency waves such as natural flood waves, and the factor 5.5 can be increased depending on the specific characteristics of the system being considered.

Each reach is described by a length, one or many Manning's coefficients of roughness, an estimate of the space increment and a table of hydraulic parameters (total top width, core top width, core area, wetted perimeter, and total area, at different water depth or elevation) for each input cross-section. The approximate space increment is used by the model to compute the exact value Δx for the reach, according to the following relationship:

$$\Delta x = \frac{\text{Total Length}}{\text{Integer}\left[\left(\frac{\text{Total Length}}{\text{Approximate space increment}}\right)\right]} \qquad \text{(C-9)}$$

$$\text{Number of mesh points} = \left[\frac{\text{total length}}{\Delta x} + 1\right] \qquad \text{(C-10)}$$

One constraining feature of the model is that a minimum of 4 space increments (5 mesh points) must be used for each reach. It is recommended to always have an even number of space increments (odd number of mesh points), in order to limit the transfer from reach to reach of numerical solution differences caused by input errors or internal boundary calculations.

The hydraulic parameter tables can be manually generated. However, front-end computer programs such as STAVE, COORD1, COORD2 (Water Planning and Management Branch, 1984b), and BLOCKCOORD (Water Planning and Management Branch, 1984a) have been written in order to generate these tables directly from input cross-sectional data.

These programs allow the data to be read from processed hydrographic charts, digitized survey field sheets, topographic maps and aerial survey data bases.

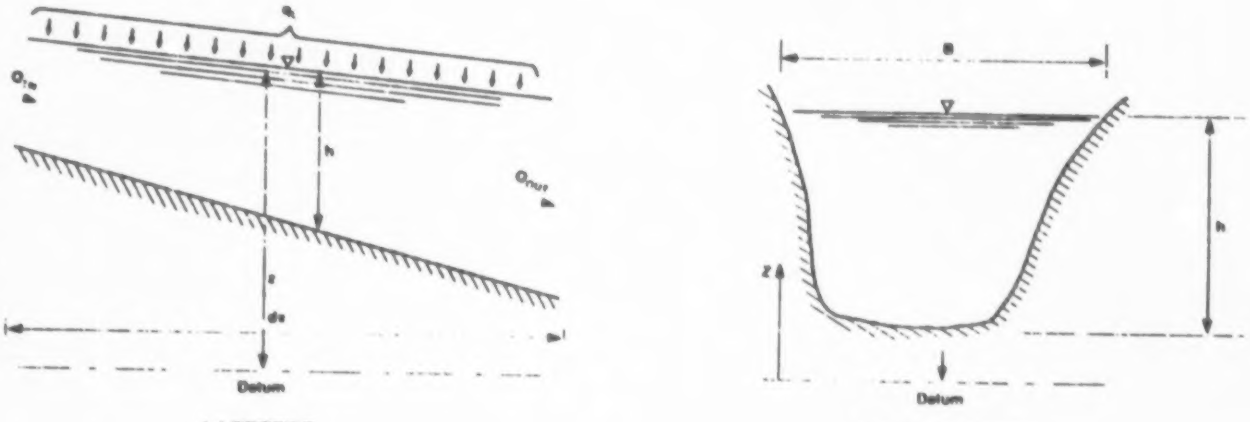
C.1.4 Description of 1-D Model Output

Three different files are created during a simulation by the 1-D model:

The first file is a printout of the results as requested by the user. These results can include: water level and discharge hydrographs that can be produced at any computational mesh point in the network; water level profiles that can be obtained in any reach and for any time (the time must correspond to an integer number of computational time steps).

A second file, known as the TAPE10 file, includes the water levels and discharges at each computational mesh point and for each time period. This file is saved to be later used to provide additional information that was not requested in the printed output. For instance, this file can be used as an input file for a plot program or to produce daily averages at a specified location.

The last file created by a computer run of the model is a TAPE17 start-up file. It includes the water levels and discharges at each mesh point for the last time step calculation. This file is used as input initial conditions for the following simulation period.


q_l
Q_in
Q_out
h
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Datum
B
Z
h
Datum

(a) PROFILE

(b) CROSS-SECTION

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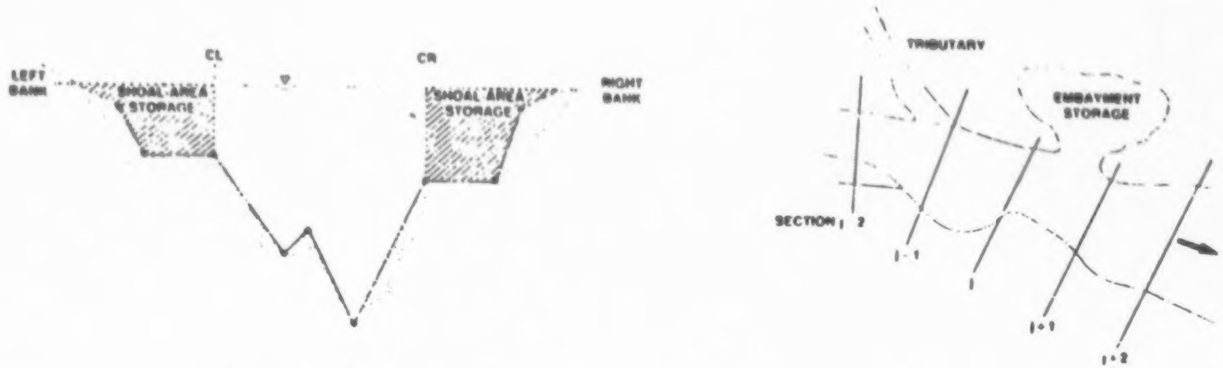
REACH PROFILE AND CROSS SECTION

FIGURE C.I


LEFT BANK
SHOAL-AREA STORAGE
CL
▽
CR
SHOAL-AREA STORAGE
RIGHT BANK
TRIBUTARY
EMBAYMENT STORAGE
SECTION i - 2
i - 1
i
i + 1
i + 2

(a) SHOAL-AREA STORAGE

(b) EMBAYMENT STORAGE

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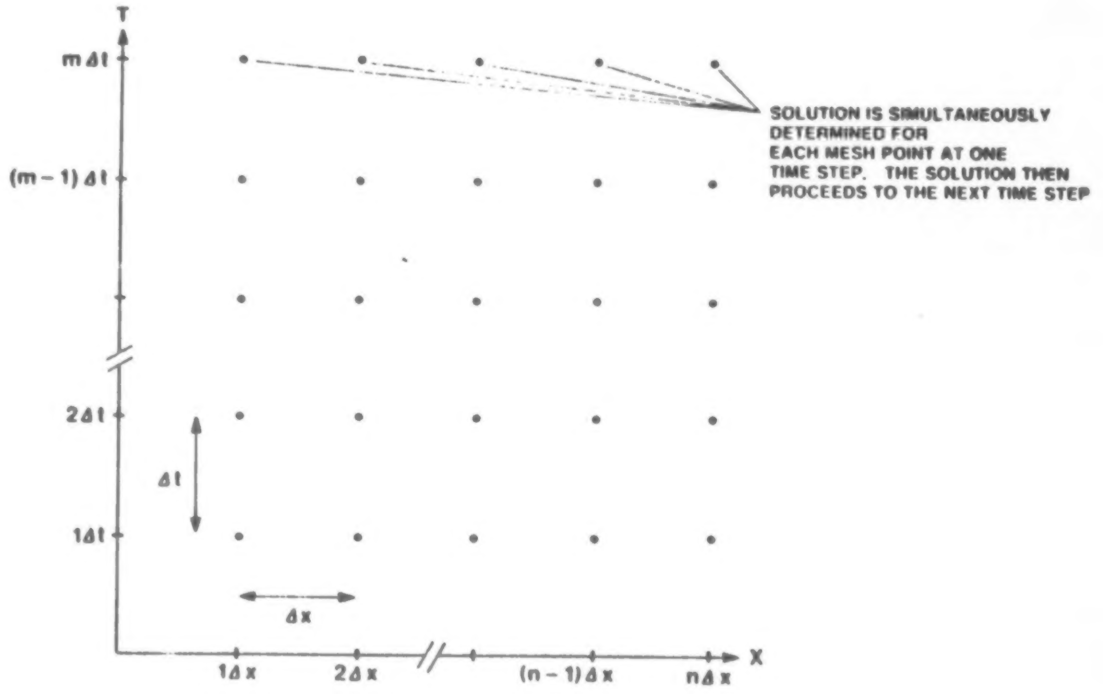
LEGEND

PHYSICAL OFF-CHANNEL STORAGE

FIGURE C.2


T
m Δt
(m − 1) Δt
2Δt
1Δt
Δt
Δx
1Δx
2Δx
(n − 1) Δx
nΔx
X
SOLUTION IS SIMULTANEOUSLY
DETERMINED FOR
EACH MESH POINT AT ONE
TIME STEP. THE SOLUTION THEN
PROCEEDS TO THE NEXT TIME STEP

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DISCRETIZATION OF SOLUTION

FIGURE C.3